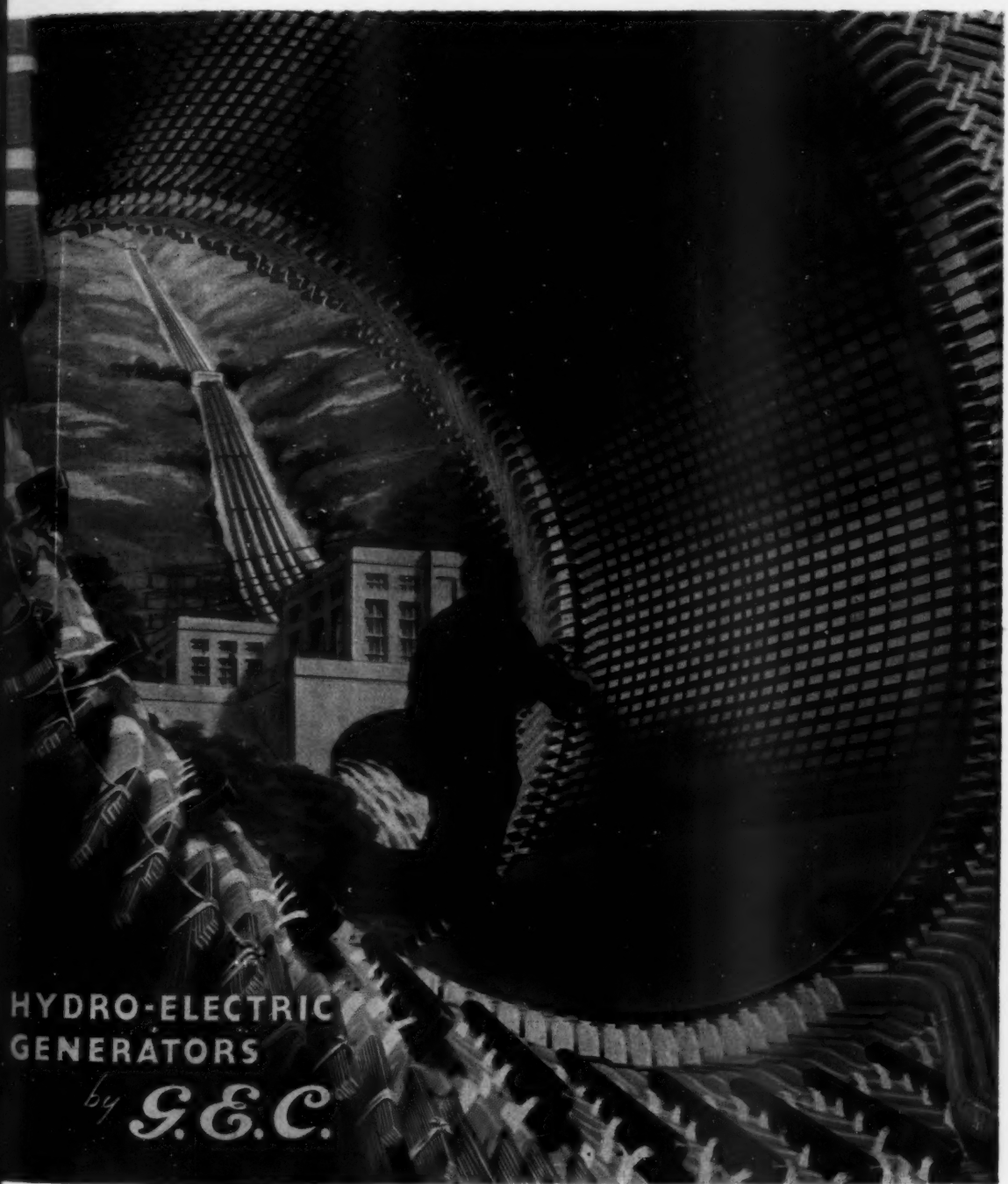
Water Power

INTERNATIONAL DEVELOPMENT OF HYDRO-ELECTRIC POWER

PRICE 2/6 SEPTEMBER 1957



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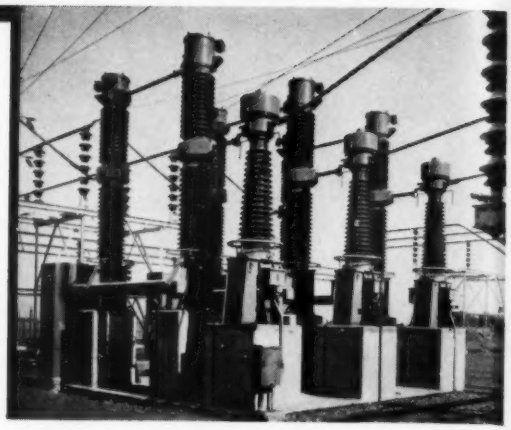

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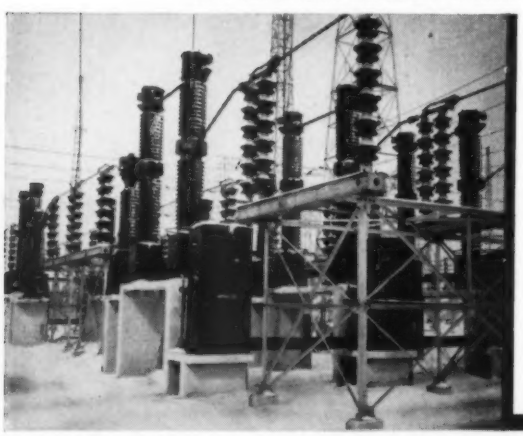

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WATER POWER

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Volume 9 Number 9

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Research in Portugal

ONE of the highlights of the recent Lisbon Congress of the International Association for Hydraulic Research was a tour of inspection of the Laboratório Nacional de Engenharia Civil in which the Congress was held. The tour proved something of a revelation, in respect both of the extent of the equipment available and of the quality of the work being done, and the general feeling amongst the Congress members—and particularly amongst those from countries taking a leading part in hydraulic research—was one of admiration.

The National Civil-Engineering Laboratory is one of the main departments of the Portuguese Ministry of Public Works, and investigations concerning the construction of public works of every nature, municipal civil-engineering undertakings, and large independently built structures such as dams, are all channelled through this laboratory. Apart from the obvious economic advantages of this concentration of activities, it is possible to co-ordinate all the technical aspects of any given problem, and to view any proposed investigation from the national standpoint. The laboratory commenced its activities about ten years ago, and the present magnificent premises, on the outskirts of Lisbon, were built about five years ago and are capable of considerable enlargement.

In addition to the administrative and general technical offices, the organisation, under Mr. Manuel Rocha, is divided into three main departments. The first deals with construction methods, standardisation, and documentation; the second covers structures, dams, wood, foundations, roads and airfields, physics of buildings, fluvial hydraulics, and maritime hydraulics; and the third is concerned with constructional materials, including cement and concrete, ceramics, and metals, and with chemical investigations.

Much could be said about any or all of these sections, but we should particularly like to refer to the brilliant work on dam design being carried out under the direction of Mr. J. Laginha Serafim, whose study of uplift pressure in dams has already been noticed in these columns.* Under Mr. Serafim Portugal has built a number of beautiful concrete dams of the arch-gravity and, more recently, of the double-curvature arch types.

Models for stress analysis constitute an essential tool in the investigation of all medium and large-sized dams; they are usually tested in duplicate, and for some dams as many as six models have been tested. Model testing, of course, is now a normal procedure with progressive dam designers, but in certain respects we believe the methods adopted at this laboratory to be unique.

Of these methods we would give pride of place to a technique of determining the ideal shape of an arch dam by means of a rubber model in which all stresses are reversed. A rubber membrane is made in the shape of the proposed dam, and is loaded on the downstream side by water. The weight of the dam is represented in reverse by a series of strings rising vertically to pass over pulleys and attached to weights. In this condition the membrane assumes a shape in which the primary internal stresses are purely tensile, which corresponds to a state of pure compression in the actual dam. This shape is recorded photogrammetrically, as it represents the optimum shape of the dam.

This method of investigation has confirmed that the circular horizontal arch is not the most economical shape but that the arch should be elliptical. A practical approximation to this is obtained by an arch comprising three circular arcs, the central one having the smallest radius. We were informed that in one dam design on these principles, 270,000 cu. m. of concrete had been saved on a total volume of 1,100,000 cu. m. without increasing the stresses in the dam.

A second investigation on which the Dams Section of the laboratory has done pioneer work is that of temperature stresses in concrete dams. In this connection a plaster model is used which is placed under oil on both upstream and downstream sides. Heating

* Mr. Serafim's treatise, "A Subpressao nas Barragens," was reviewed in WATER POWER, February 1955, p. 44.

and cooling coils in the oil reproduce the temperature and temperature gradient of the water, and the temperature of the air and sun on the downstream side, and the resulting stresses are observed by the usual methods. By these and other means it has been ascertained that thermal stresses as high as 40 kg. per sq. cm. can exist in an actual dam, and this added to a design stress of, say, 70 kg. per sq. cm., can give a total stress of the order of 110 kg. per sq. cm.

These conclusions have led in turn to a programme of investigations into the behaviour of concrete at elevated temperatures, particularly in respect of creep and of change in modulus of elasticity.

Hydraulic models of dams are also tested to study spillway conditions, and we saw in operation two hydraulic models of which we had already seen the prototypes under construction—an unusual design of dam at Picote, and a glory-hole spillway at Paradela.

Space precludes our more than mentioning the comprehensive instrumentation of the actual dams so that their behaviour can be compared with the design predictions, and the work on the properties and behaviour of rock foundations.

We had the privilege of visiting all the major modern hydro-electric schemes in Portugal—completed and under construction—and hope to describe some of them in due course. In the meantime, we are happy to record our respect for the achievements of Portuguese engineers in the hydro-electric field, and our appreciation of the contribution that has been made to their accomplishments by the National Civil-Engineering Laboratory.

The IAHR Congress

THE IAHR Seventh Congress, which we have briefly mentioned above, was admirably organised by Laboratório Nacional de Engenharia Civil, and, having regard to the prevailing temperature in Lisbon, was mercifully held in the air-conditioned amphitheatre of the laboratory. The papers were weighty in both senses, for they flowed, for the most part, in deep mathematical waters, and they provided a problem of excess luggage for the airport officials. The one disappointing feature was the lack of discussion, due, no doubt, to the fact that the papers were not available until the commencement of the Congress. This, by the way, was not the fault of the organisers, but arose from the fact that only a minority of authors sent in their papers by the prescribed date. However, this shortcoming was largely compensated by an off-the-record session devoted to a discussion of cavitation problems. We hope to publish a brief commentary on some of the proceedings in early issues.

The Eighth Congress is to be held in Quebec in 1959, probably in the September, and will discuss the following subjects: (1) Hydraulics of gates and valves, including problems of vibration, cavitation and uplift; (2) Hydraulics of locks; (3) Density currents; and (4) Air entrainment, including air vents.

The Derbendi Khan Project, Iraq

THIS project is designed to serve several purposes—power production, irrigation and flood control. To enable the rockfill dam to be built two diversion tunnels have been driven, one having a diameter of 9 m. and the other 6 m. Between them they will pass a maximum flow of 1,670 cu. m. per sec. These tunnels will ultimately be used for power and irrigation purposes but the upstream ends will be plugged and the water introduced through two inclined shafts from the intake tower.

The initial development will contain the outlet works equipped with three 8 ft. diameter regulating valves of the Howell-Bunger type and two small 800 kW generating sets to provide service power. Ultimately three 37,500 kW generating sets will be installed. A chute-type spillway will be provided, controlled by three radial gates.

When finished the dam will be 400 ft. high and is being built by Derbendi Khan Contractors, a joint venture of J. A. Jones Construction Company, Carolina; Tecon Corporation, Texas; The Chas. H. Tompkins Company, Washington, D.C.; and Beton und Monierbau A.G., Dusseldorf. Contracts for the intake gates, gantry crane, steel penstocks and tunnel linings have been awarded to the Fried Krupp and Dortmunder Union concerns acting jointly. The consulting engineers are the Harza Engineering Company, of Chicago.

Roxburgh Turbines

THE British Thomson-Houston Co. Ltd. have been awarded a contract valued at £800,000 covering four further vertical hydro-electric generators for Roxburgh power station, New Zealand. The machines are each rated at 44,444 kVA, 40,000 kW, 11 kV, 136 r.p.m. They will be exactly similar to the four already supplied by this firm, which were put into commission during the latter half of 1956. The turbines for driving the new generators will be supplied by Dominion Engineering Co. Ltd., of Canada, who also manufactured those for the first four machines. Roxburgh power station, with an ultimate installed capacity of 320,000 kW, will, when completed, be the largest in New Zealand and will be of inestimable help in satisfying power demands for the following three or four years. When the 500,000 V direct-current submarine power cable is laid across Cook Strait, thus linking the power systems of the North and South Islands, Roxburgh will play an even more prominent part in New Zealand's overall economy, as the shortage of power is more acute in the North Island, which is more highly industrialised than the South Island. On the other hand the South Island has by far the larger water-power potential.

New Dam for Brazil

GOOD progress is being made with the Camargos dam which is being built on the Rio Grande river by Companhia Morrison-Knudsen do Brazil, S.A. It will be 110 ft. high and have a total crest length of 1,800 ft., including the spillway section of 1,100 ft. Some 212,000 cu. yards of concrete will be needed and 400,000 cu. yards of rock and earth fill for an adjoining embankment. Rock is quarried about three miles away and crushed to five sizes (6 in. maximum) in a plant on site, and the concrete is mixed in a Johnson semi-automatic batch plant equipped with two 2 cu. yard mixers. The earth section of the dam is being built by tractors and carrying scrapers and has gone rapidly ahead once the summer rains ceased in March.

The Camargos dam is being built for Companhia

de Eletricidade do Alto Rio Grande, a subsidiary of S.A. CEMIG, with the purpose of serving as a storage and regulating dam for the Itutinga station, which was described in our January 1954 issue, and lies downstream of the new dam. This station will ultimately have a capacity of 62,000 kW and the Camargos development will also be equipped with two 24,000 kW sets, but as yet only the foundations of the outdoor-type power station will be built.

Shiroro Gorge, Nigeria

THE Northern Region Development Corporation of Nigeria has allocated £500,000 for the purchase of share capital in a company to be formed for the purpose of developing hydro-electric power in the Shiroro Gorge area of Northern Nigeria.

The money has been allocated on the understanding that the proposed company is to consist of the Electricity Corporation, the Federal Government, the Northern Regional Government and the Northern Region Development Corporation. An annual examination is to be made of the position at a full meeting of the Corporation at which it will be decided, in accordance with the circumstances prevailing, whether this sum should be withdrawn, reduced or increased.

Earlier this year a firm of British consulting engineers began investigations into the possibility of damming the Shiroro Gorge, some 55 miles from Kaduna, the Northern Regional capital, to provide electrical power for the new industrial areas at Kaduna and Zaria. It has been estimated that the Kaduna river is capable of producing over 400 million kWh of electrical energy in a year of average rainfall. To provide sufficient storage for power to be generated during the dry season a dam of considerable height would be necessary.

New Hydro-Electric Plant for Jamaica

THE Jamaica Public Service Co. Ltd. have awarded the contract for a hydro-electric generating plant, to be installed in Maggotty Falls power station, to The Harland Engineering Co. Ltd., of Alloa, Scotland. The "Harland-Morgan Smith" hydraulic turbine will be of the Francis type and rated at 8,400 h.p. when operating under a net head of 290 ft., and be vertically coupled to a 7,500 kVA "Harland" synchronous generator. The unit is specially designed for operation at an initial frequency of 40 cycles, with provision for easy conversion to 50 cycles in the future. Arrangements have been made for fully-automatic local and remote control, which had been previously adopted at the Public Service Company's Lower White River and Rio Bueno generating stations. These two earlier installations were also equipped with Harland equipment.

Electricity Supply in Ireland

THE Thirtieth Annual Report of the Electricity Supply Board of Ireland, in referring to hydro-electric matters, states that the rainfall for the year ended March 31, 1957, was above average and 689 million kWh were generated, as compared with 590 million for an average year. This increase resulted in a saving of £285,000 on fuel charges.

Further works for improving the drainage and power possibilities of the river Erne have now been completed and include the dredging of the river channel between Belleek and the Lower Lough Erne and the construction of a new weir at Portora for regulating the water level of Upper Lough Erne. The two dams at Inniscarra and Carrigadrohid, on the river Lee, are now virtually completed and both reservoirs are in process of filling. The Inniscarra station is equipped with one 15,000 kW and one 4,000 kW set, while the Carrigadrohid station has one 8,000 kW set; both will be put into commission before the end of this year. A further hydro-electric station at Clady, on the river of that name in Donegal, is not so far advanced, but it is hoped to have the 4,000 kW set installed there in 1958.

Major New Transmission Lines

THE New South Wales Electricity Commission has let contracts for the erection of two major transmission lines which will cost some £3 million and will form sections of the 330MV "backbone" of the New South Wales bulk supply system. One of the new lines will extend from Muswellbrook to Tamworth, and the other from Yass to the major hydro-electric stations of the Snowy Mountains scheme.

Design, planning and survey work for the new lines was undertaken by the Commission, and contracts for their erection have been let to the Electric Power Transmission Co. Ltd. The Muswellbrook-Tamworth line will be 80 miles long, and though used initially at a lower voltage, will be strung for 330 MV operation using twin conductors each about 1 in. in diameter. The 90 mile Yass-Upper Tumut line will interconnect the Snowy Mountains hydro-electric stations with the Commission's system at Yass, and in turn with the major thermal power stations.

The Commission's contract includes the erection for the Snowy Mountains Authority of the section of line between the cableyard and the switching station, and the supply of towers for the high-altitude line which will carry power to Victoria.

Errochty Power Station

THE RT. HON. JOHN S. MACLAY, C.M.G., M.P., Secretary of State for Scotland, officially opened the North of Scotland Hydro-Electric Board's Errochty power station on July 27. This 75,000 kW station is the fourth to be built in the Tummel Valley, and the 26th main hydro-electric station to be completed by the Board since the war.

Mr. Maclay stated that at the end of 1948 the total generating capacity installed in electricity undertakings in Scotland was under 1,200 MW. At the end of last year it was over 2,000 MW, an increase of 83%.

The chairman of the Board, Mr. Thomas Johnston, stated that last year the generation over all the Board's hydro schemes represented a coal saving of 715,000 tons. At a value of £7 per ton, the saving meant over £5 million, or about 7% on the total capital of £70 million expended on the schemes.

Sir Donald Perrott, K.B.E., Member of the U.K. Atomic Energy Authority, forecast the construction of more hydro-electric schemes in the North of Scotland to meet the need for pumped storage.

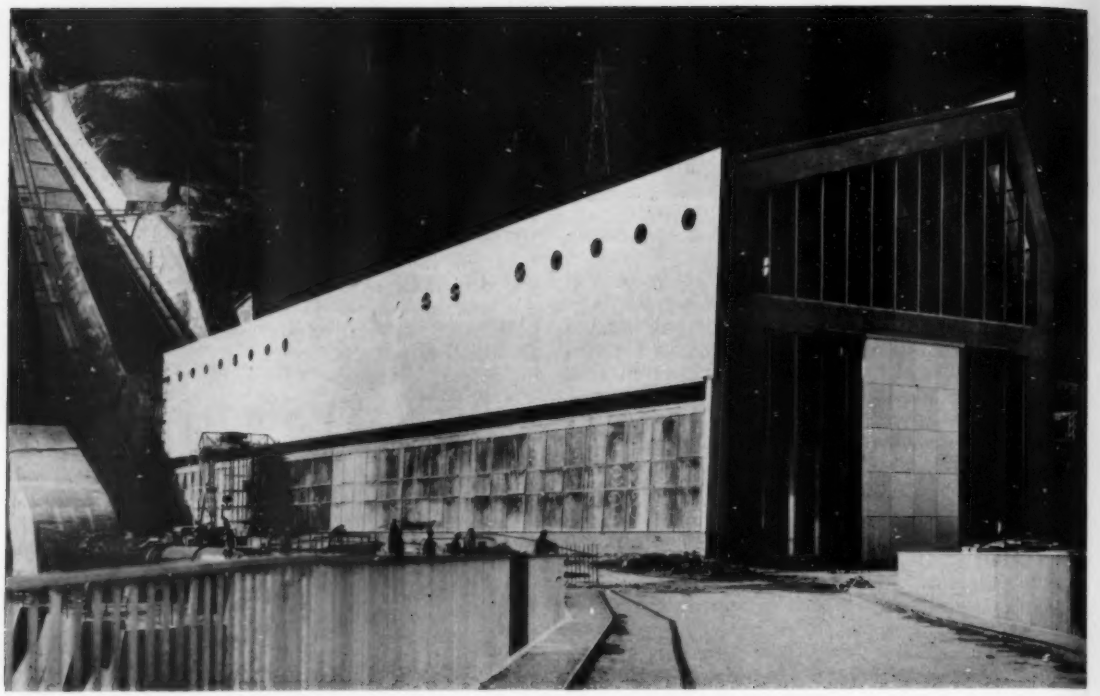


Fig. 11. Exterior view of Cimego power station

The Alto Chiese Development

We devote this final article to a description of the Cimego station, which contains what are at present the largest horizontal two-wheel single-jet Pelton sets in the world

PART TWO

IN our previous article we gave an outline of the Alto Chiese development and described the major civil-engineering works. The scheme comprises four reservoirs and three power stations. The reservoirs include Malga Bissina (60 million cu. m. at 1,788 m.), Malga Boazzo (11·8 million cu. m. at 1,224·5 m.), Ponte Murandin (290,000 cu. m. at 717 m.), and the Cimego tailpond at a level of 486 m. The first power station, at Boazzo, (95 MW) is fed from the Malga Bissina reservoir; the second, at Cimego, (229 MW) is fed from the Malga Boazzo and Ponte Murandin reservoirs; and the third, at Storo (18 MW), draws from the Cimego tailpond.

When we visited the development the Boazzo and Storo stations were in the early civil-engineering stages, and there is little that we can add to the facts already given, but the Cimego station had recently been brought into commission, and we propose to describe it in some detail.

Exterior and interior views of the Cimego station are reproduced in Figs. 11, 13, 14 and 15, and a plan is given in Fig. 12. An interesting feature is the unusual and attractive architectural design of the building, a clear impression of which can be gained from the illustrations.

As will be seen from Fig. 13 the station is supplied by two external penstocks, one conveying the supply from Malga Boazzo reservoir and the other that from Ponte Murandin.

In our previous issue we referred briefly to the surge chamber at the end of the Boazzo-Cimego main supply tunnel and the drawing, Fig. 16, shows the design in greater detail. Immediately downstream of the surge chamber is the inlet of the steel penstock 3·3 m. in internal diameter, and the valve chamber in which are located an automatic butterfly valve, an automatic air valve, and also the winch for the penstock service trolley. Thence the pensock descends a

total distance of about 1,220 m. to the station, reducing progressively in diameter to 2·9 m. The pipeline is of hooped construction and is secured by ten anchor blocks, there being an expansion joint immediately below each block. At the station the penstock is carried alongside the building, as seen in Fig. 14, a branch being taken off for each of the four turbine nozzles. The maximum operating head is 739 m. and the maximum flow 34 cu. m. per sec.

In view of the special requirements of the station, research investigations were undertaken to determine the dimensions of the penstock to ensure the hydraulic and mechanical stability of the system.

The penstock and valves of the Boazzo diversion were supplied by Acciaieria e Tubificio di Brescia.

The Ponte Murandin penstock, manufactured by Officine Bossi, is laid alongside the main penstock on the opposite side of the trolley track. It is 462 m. long and is a plain steel pipe ranging in internal diameter from 1·35 to 1·15 m.

The butterfly valve was supplied by A.T.B., Brescia.

Main Generating Sets

As explained in our previous article, Cimego was designed as a frequency-control station, and to meet this requirement two sets each of 110 MW capacity were chosen. Each machine is a two-bearing assembly consisting of a horizontal alternator with a single-jet impulse wheel overhung at each end, and the two sets are believed to be the most powerful of this type at present in service in the world. Drawings of one of the machines are reproduced in Figs. 17 and 18.

Turbines

The turbines were built by Costruzioni Meccaniche Riva, Milan. Each double turbine at rated full load develops 150,000 h.p. at 300 r.p.m., discharging 17·75 cu. m. per sec. under a head of 721 m. but at full needle opening 155,000 h.p. can be sustained.

Each runner (Fig. 19) weighs 45 tons and has a diameter of 4·47 m. In order to avoid excessively difficult casting problems, which would arise if runners of this size were cast in one piece, each runner has been cast in three segments each consisting of seven buckets in 13% chrome stainless steel. These three segments were then secured to a forged-steel central disc by means of tapered bolts forced into their seats by a special hydraulic press, this operation being controlled by strain gauges.

Fig. 20 is a view of a turbine in the works showing the straight-flow nozzle which is a feature of Riva designs*, and which can be seen in cross section in Fig. 17. Compared with the conventional elbow-type nozzle, the straight-flow design ensures the uniformity and axial symmetry of the flow, these being the most important factors in securing the homogeneity and cylindricity of the jet. All this affects the hydraulic efficiency of the turbine, because it influences the conversion process from pressure to kinetic energy of the jet; furthermore, the working conditions of the jet on the runner are stated to be improved, with cor-

* See WATER POWER, April 1952, p. 155.

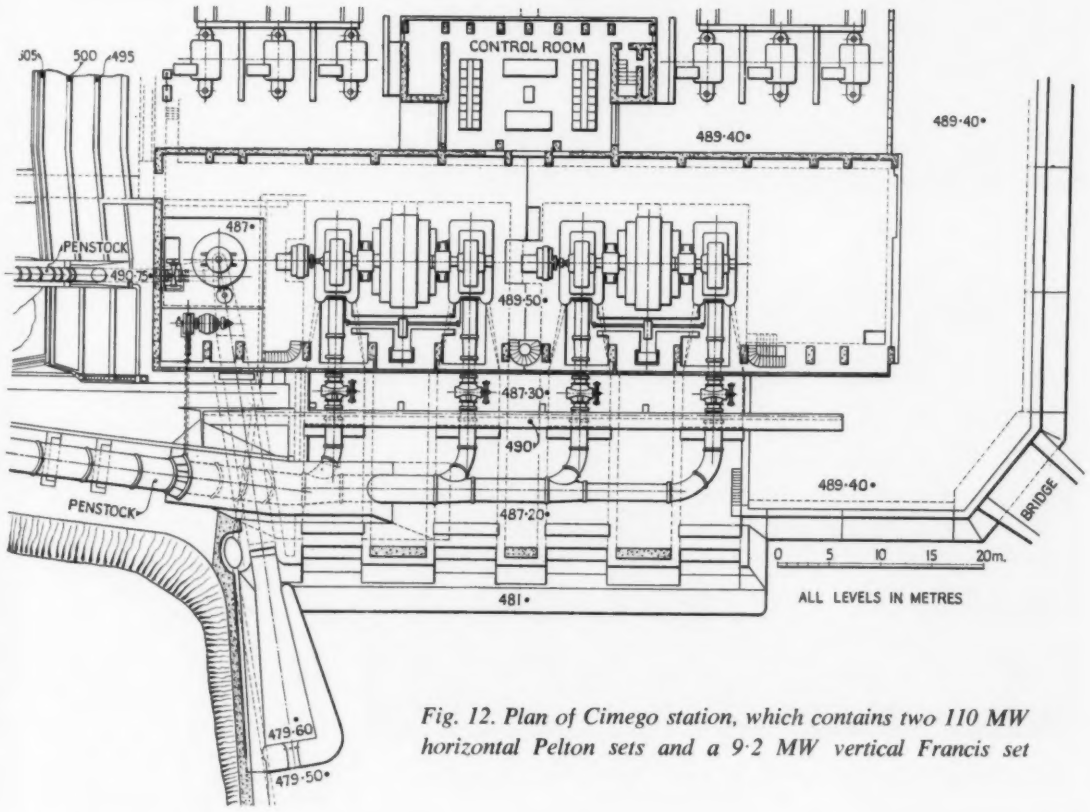


Fig. 12. Plan of Cimego station, which contains two 110 MW horizontal Pelton sets and a 9·2 MW vertical Francis set

Fig. 13. General view of Cimego station, showing the penstocks and the switchyard

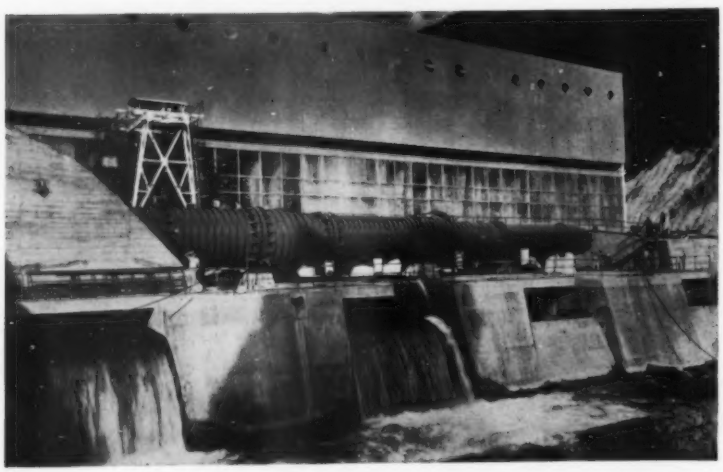


Fig. 14. Side view of station, showing main penstock and turbine discharges

responding benefit to the life of the runner.

The reason for the elbow in the conventional nozzle, of course, is to permit the connection between the needle and its external operating gear, and the Riva design owes its practicability to the success with which the needle operating gear has been housed within a streamlined cylindrical body concentric with the external pipe and held in place by two diametral wings.

Referring to Fig. 17, four servomotor pistons are formed integrally on the stem of the needle, the oil spaces in front of these pistons being interconnected so that the total operating force exerted on the needle is that corresponding to the sum of the piston areas. This servomotor system operates only in the opening sense; closure is effected by the hydraulic closing tendency of the needle, assisted by a spring. Thus, the needle is opened by feeding oil under pressure to the servomotor, and is closed by releasing the oil. During closure the speed of release of the oil is controlled by an adjustable restriction in the oil pipe, so that the servomotor acts as a brake to prevent the needle from closing faster than a predetermined rate.

Admission and release of the oil are controlled by a valve mounted on a floating lever, one end of which follows a cam actuated by the governor gear whereas the other end follows a ramp on the needle stem through the intermediary of a tappet rod. This arrangement, which can be seen in Fig. 17, results in the needle assuming a position strictly corresponding to the governor position. The governor-controlled spindle which carries the cam is also linked to the deflector, which therefore also follows the governor position.

The oil duct to the needle servomotor, the tappet rod and the necessary drain channels are all accommodated in the diametral wings. Seeing that the servomotor is oil operated it requires virtually no maintenance. The only parts subject to wear are the needle tip and the nozzle mouth ring, both of which can be replaced without disturbing the servomotor.

The governor is of the type responding to both speed and acceleration. It is incorporated with the oil pumps and accumula-

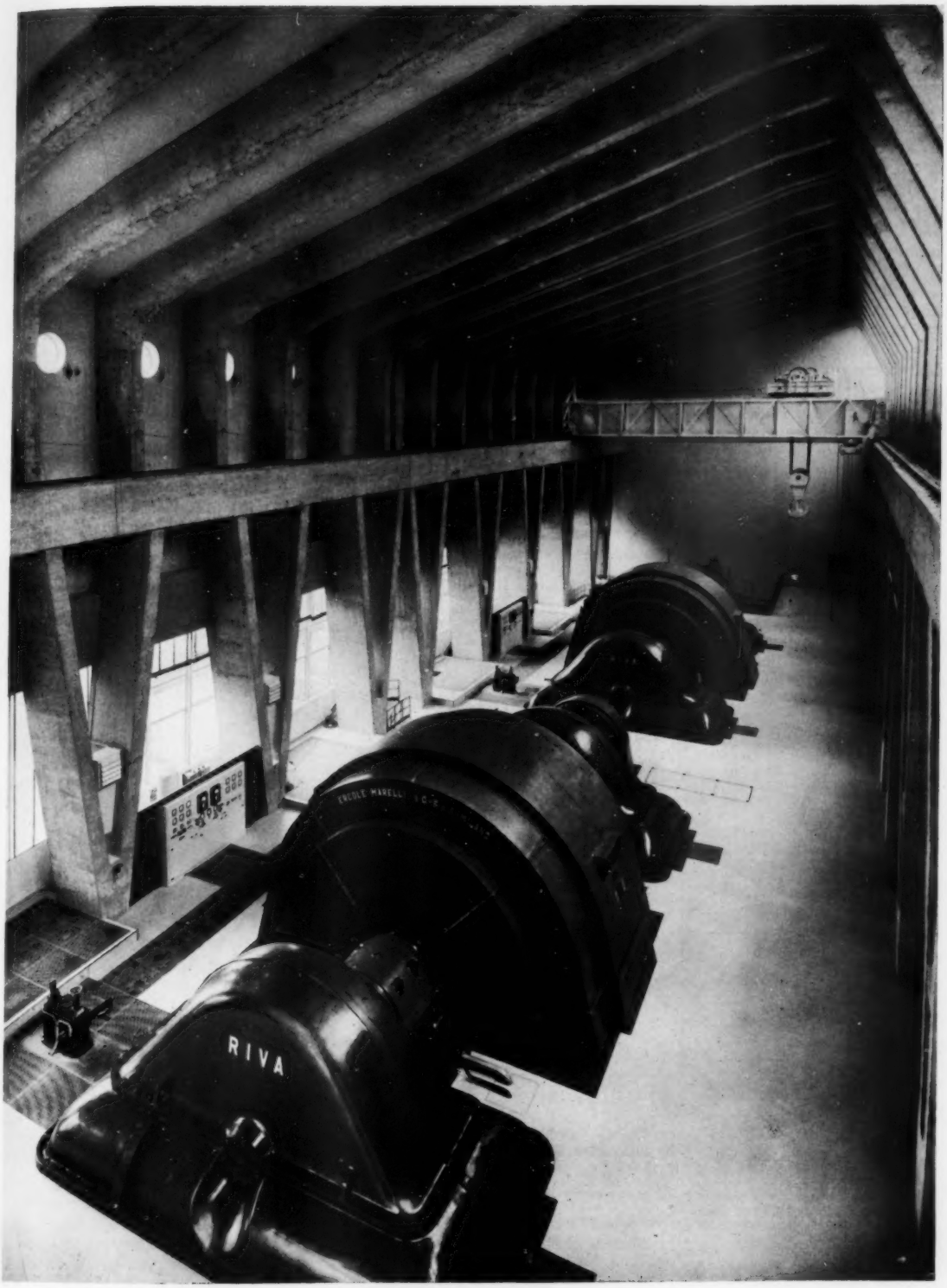


Fig. 15. Interior of Cimego station, showing the two 110 MW horizontal Pelton sets, and in the distance, the 9·2 MW Francis set

tor in a single control unit which carries all the controls and instruments necessary for manual and automatic operation and protection of the set. Starting is fully automatic from the starting of the auxiliaries to running at no load under the control of the speed governor ready for synchronising. The starting sequence can be initiated either directly from this control unit or remotely from the control room.

Oil is stored in an accumulator fed by two pumps, one acting as standby. Both are electrically driven, and one pump is provided with a small Pelton wheel in addition to the motor. Changeover from one pump to the other, and to Pelton wheel drive when necessary, is automatic. An air compressor maintains an air cushion in the upper part of the accumulator.

The turbine main valves are also of Riva construction, and are of the rotary type presenting a smooth bore when fully open. They are operated by hydraulic pressure from the penstock and are remote controlled from the control unit described above.

All possible mishaps are guarded against by a series of trips which destroy the oil pressure in the system so that the needles close under the influence of their springs, and when required, the main valves are also automatically closed.

The discharge chambers below the turbines emerge from the station immediately below the penstock platform, and, as is seen in Fig. 14, each is closed by a rectangular weir for measuring the discharge.

Alternators

The alternators, which are believed to be the largest horizontal waterwheel alternators in the world, were built by Ercole Marelli & Co., S.p.A., and one of them is seen in course of erection in Fig. 21. Each machine is rated at 110 MW, 0·9 p.f., 50 c.p.s., 13·8 kV at 300 r.p.m., and has a short-circuit ratio of 1·25. Here are some further data:

WR^2=1,160 ton m²
Inertia constant H=5·06
Transient reactance X_d'=0·284 p.u.
Subtransient reactance X_d''=0·222 p.u.
Total losses at full load, 0·9 p.f.=2,010 kW, guaranteed
=1,759 kW, measured
Efficiency at full load, 0·9 p.f.=98% guaranteed
=98·43% measured

The losses have been measured separately, by running the main exciter as a d.c. motor, fed by a converter set.

The rotor is composed of three parts, a central cast-steel bush and two shaft sections. The rotor rim is made up of circular forged steel plates which are shrunk on the bush. The field poles are jointed to the rotor rim by means of double dovetails. The rim plates and the field poles were assembled on site, as the

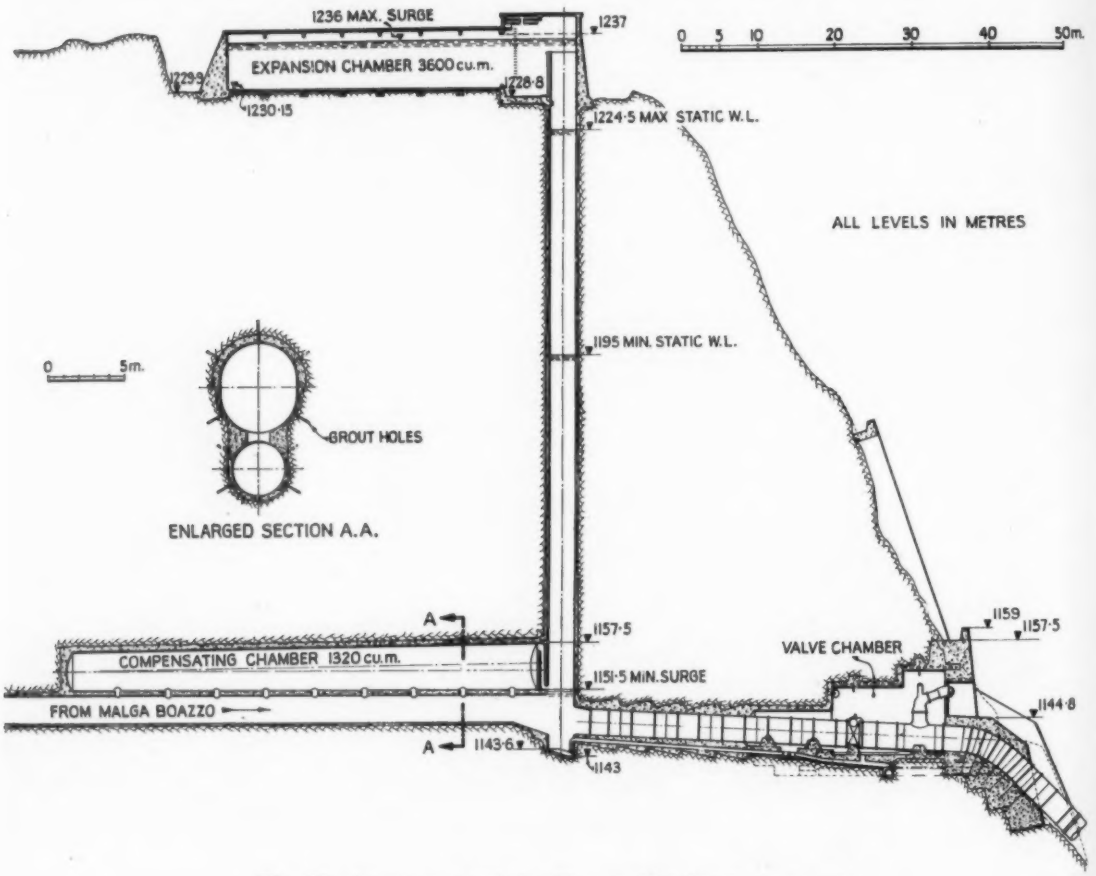


Fig. 16. Arrangement of the Cimego high-head surge tank

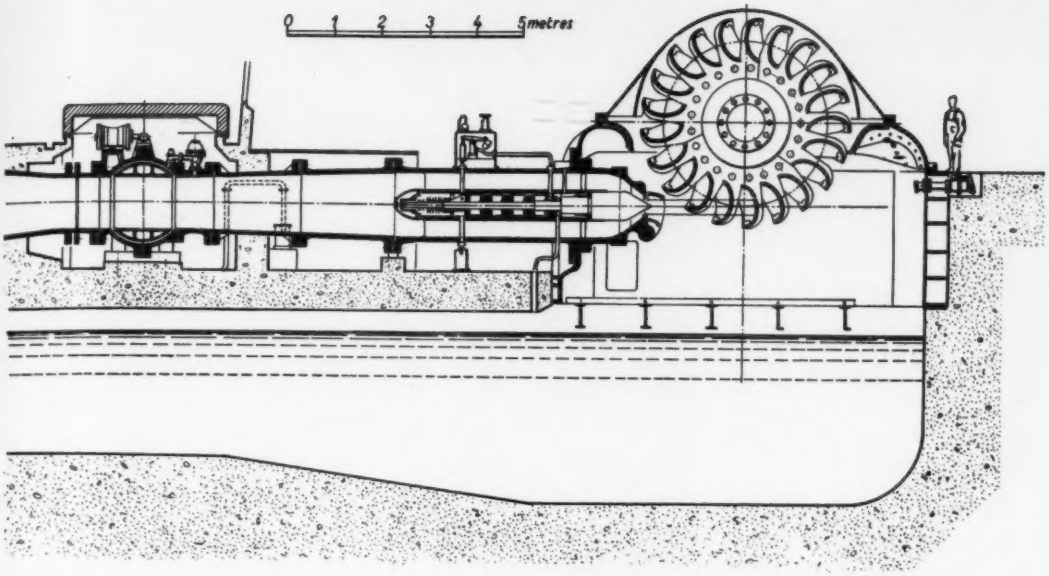


Fig. 17. Cross section through one of the Riva turbines and straight-flow nozzle

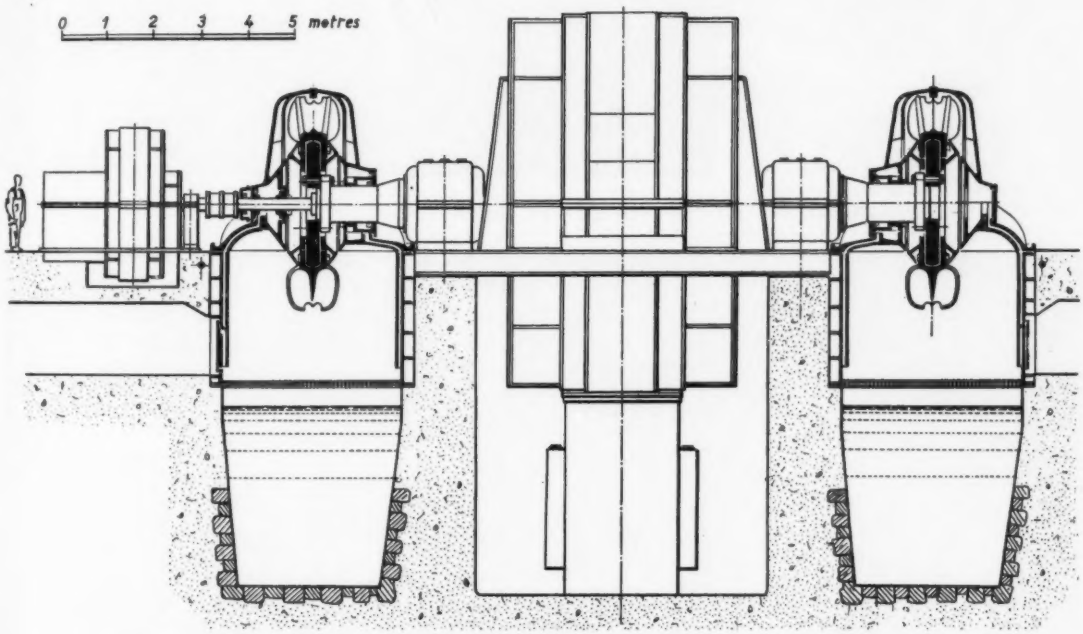


Fig. 18. Longitudinal part-sectional elevation through Riva/Marelli 110 MVA set

maximum shipping weight had to be less than 115 tons (total rotor weight 380 tons).

Westinghouse Thermalastic insulation, which is widely used and manufactured by Marelli, is used on the stator. The total weight of each alternator is 650 tons. The stator frames are split horizontally to assist transport and to facilitate erection.

The generators are provided with an automatic voltage regulation system consisting of a magnetic amplifier controlling an impulse exciter; this latter supplies the field of a booster exciter which is series connected to the main exciter field; this main exciter is shunt excited but is also provided with a partial series excitation to increase the response speed. Suitable feed-back circuits permit the use of greater amplification coefficients, thus ensuring a high response speed without instability. The power station has been also provided with a load-frequency control system, in view of possible exchanges of power with other systems running in parallel with the Edison network.

Francis and Auxiliary Sets

The supply from Ponte Murandin is utilised by a 9·2 MW vertical Francis set running at 750 r.p.m. and generating at 10 kV. The turbine and rotary valve are of Riva construction and the alternator was supplied by Breda Elettromeccanica e Locomotive S.p.A., Milan.

For station services a Riva/Breda horizontal Pelton set is installed, running at 1,000 r.p.m. and generating 1,250 kW at 3,150 V. It is supplied by a branch from the main penstock.

Mention of the auxiliary supply makes this a convenient point at which to mention the various water pumps. A pump room in the basement below the erection bay contains nine pumps supplied by A. Pell zzari & Figli, S.p.A., Arzignano; three serve the air coolers for the main generators and the oil cooler for the main transformers. two the coolers for the vertical generator and the three-winding transformer, two the lubricating-oil coolers, and two the fire services. No drainage pumps were provided as the water is gravity-drained; the outlet sections of the drain ducts have been carried at a sufficient distance downstream from the tailrace section to be completely free even

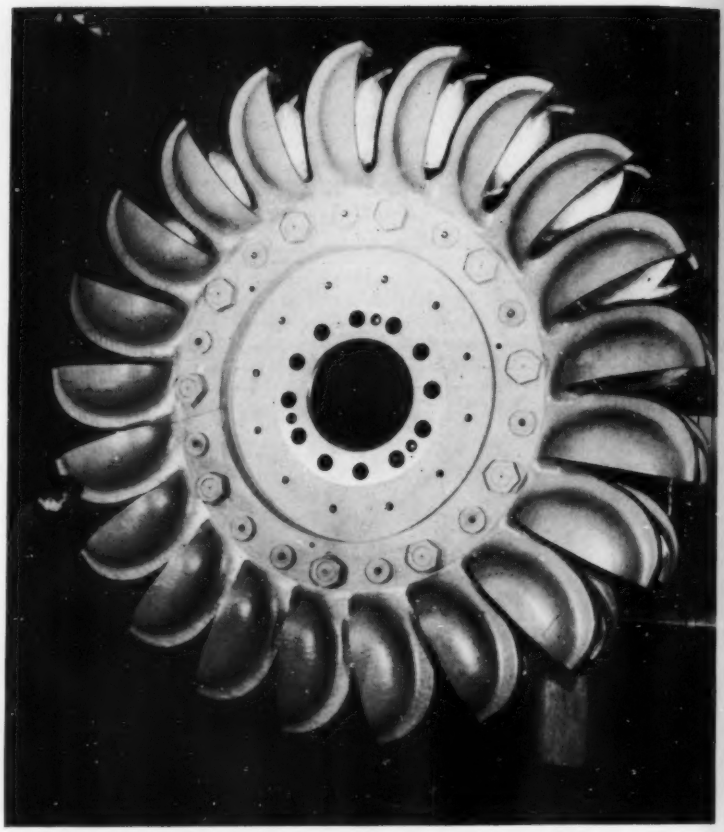


Fig. 19. One of the 4·47 m. stainless-steel Pelton wheels

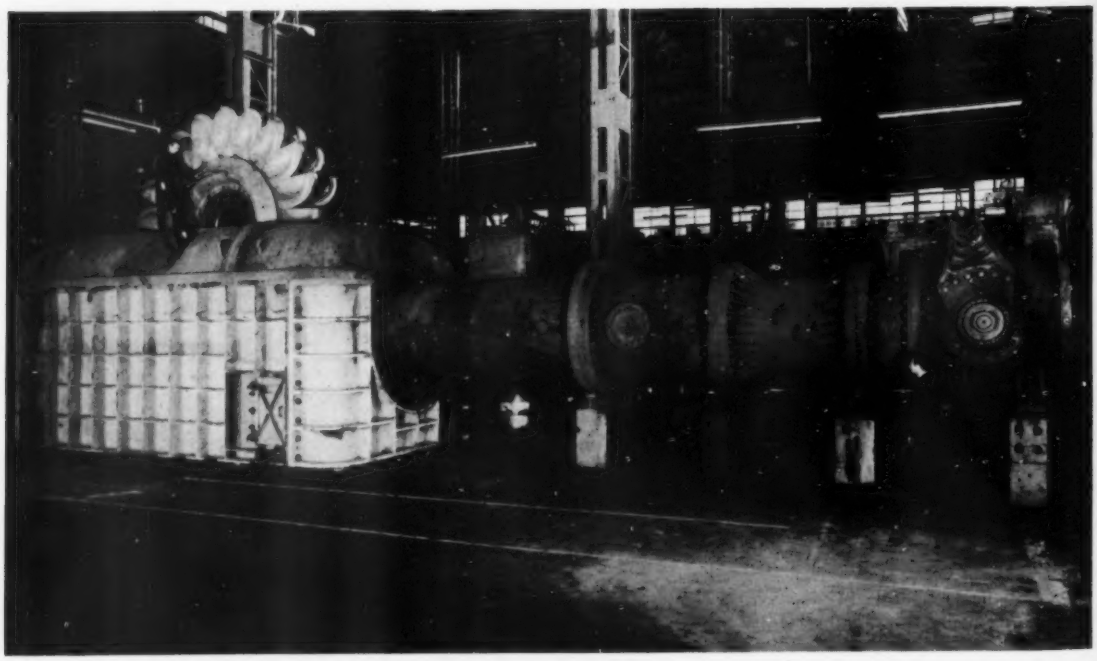


Fig. 20. One of the turbines with straight-flow nozzle in the Riva works

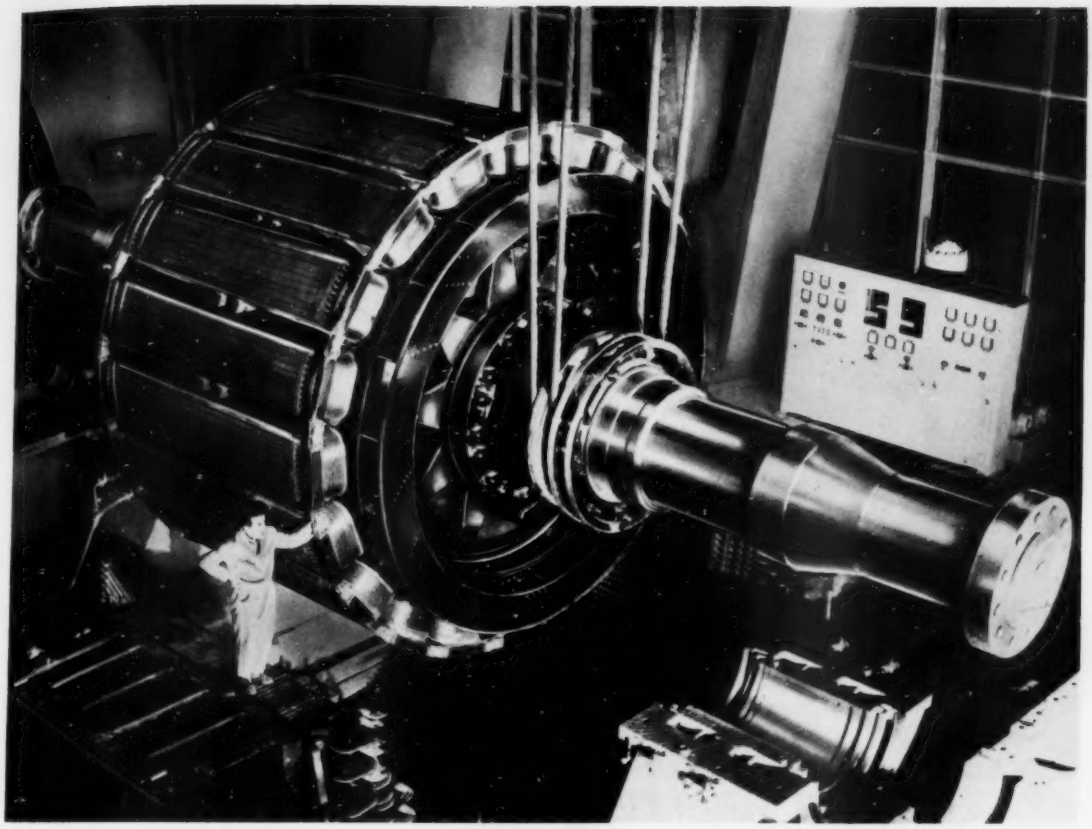


Fig. 21. Rotor of one of the 110 MW Marelli generators being lowered into place

during periods of the most exceptional high water.

Transformers

In order to simplify the transport problem and to reduce the cost of spares, it was decided that the main transformers should consist of banks of single-phase units. Seven such units have been supplied by Ercole Marelli & Co., making three units for each generator and one spare. Each unit is rated at 36,666 kVA, 13·8/138·4 kV, combining to form a three-phase delta/star bank at 110 MVA, 13·8/240 kV. Connection between each alternator and its transformer bank is through copper busbars and an isolating switch. Synchronising and load switching are effected by circuit breakers on the high-voltage side.

The cores are built up from laminations of grain-oriented steel annealed in an inert atmosphere, and the insulation is suitable for operation with earthed neutral. The acceptance tests have ascertained the following values of the main characteristics:

No-load loss at rated voltage	— 88 kW
No-load loss at 110% voltage	— 117 kW
Copper loss at full load	— 125 kW
Impedance drop	— 10%
No-load current at rated voltage	— 1·26%
No-load current at 110% voltage	— 2·43%

The specified voltage tests included 460 kV applied to the high-voltage side for one minute, 48 kV applied to the low-voltage side for one minute, and an impulse test equivalent to 1,050 kV full wave for 1/50 microsec. In addition, a special series of impulse tests to breakdown was carried out in the Marelli laboratory in the presence of members of the Edison staff. Standard impulse tests up to 1,050 kV equivalent voltage were first made in accordance with A.S.A. standards, all with successful results. The standard bushing was then replaced with a special 340 kV bushing which is part of the Marelli test-floor equipment, and impulse voltages were applied, commencing at 1,050 kV and increasing in steps of 150 kV until breakdown occurred, three impulses being applied at each voltage above the initial value. No breakdown occurred until the second test at 1,500 kV, when a failure began to reveal itself which proved final at 1,650 kV. After this the unit was subjected to ten impulses corresponding to 1,050 kV, which it survived successfully, and then to industrial-frequency tests of 442 kV for 60 sec. and 500 kV for 10 sec., with equally successful results. A test was also made to check the efficiency of the insulation to earth.

The transformer banks are arranged in outdoor cubicles alongside the station rear wall and opposite their respective generators, as may be seen in Fig. 22. The usual system of rails enables any unit to be moved to a separate transformer maintenance shop, which is served by a 105 ton Gentili crane and where all work can be carried out under cover.

Cooling is achieved by two oil-to-water heat exchangers, each capable of absorbing 70% of the total

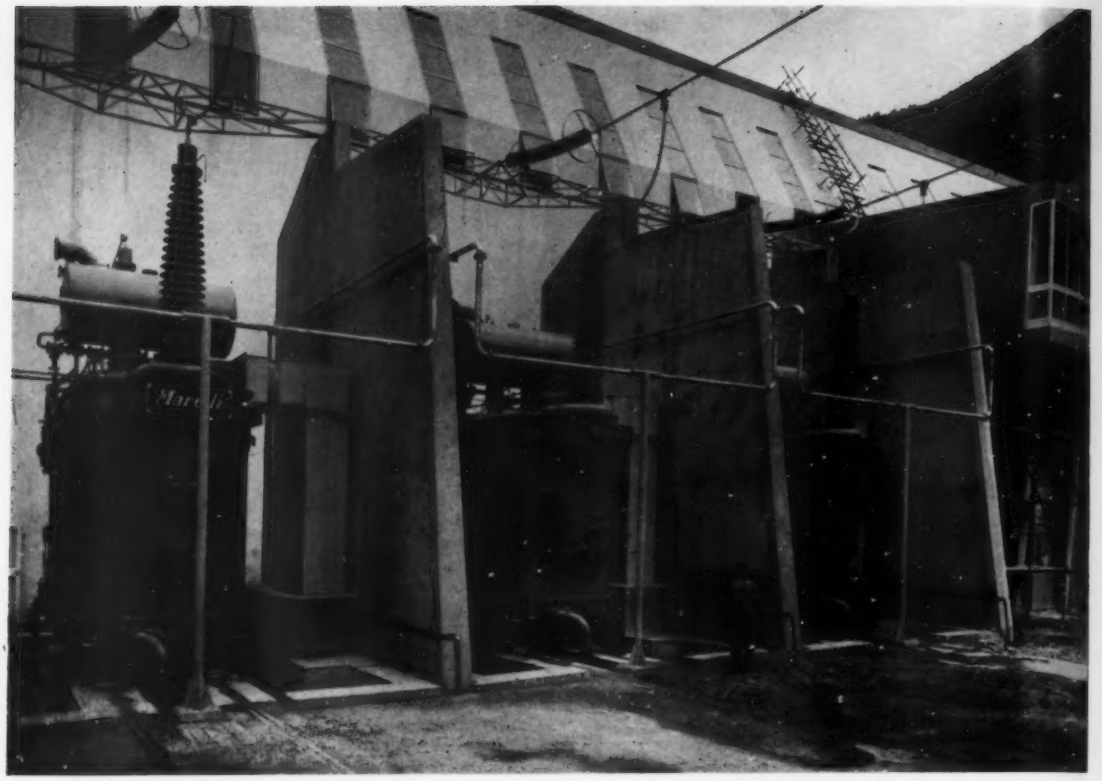


Fig. 22. A 110 MVA 13·8/240 kV bank of Marelli transformers

loss. The oil pumps are a standard Marelli design arranged to work under oil. A Mather & Platt fire protection system is installed.

Switchyard and Electrical System

The switchyard is situated immediately at the rear of the station and occupies an area that has been levelled off alongside the river. The structural steelwork was supplied by Soc. Anonima Elettrificazione, and the main circuit breakers, which are of the minimum-oil type, by Scarpa & Magnano.

A single-line diagram of the electrical system is given in Fig. 23. The main generators are connected to their respective transformer banks through segregated-phase metal-clad copper busbars of 6,440 sq. mm. cross section and through three-pole isolators of 5,000 A carrying capacity, the neutral points being earthed through a resistance. The operations of synchronising and of breaking circuit are carried out by circuit breakers immediately on the high-voltage side of the transformer banks. The power from these machines is fed to a 220 kV double-busbar system which will eventually serve eight transmission lines. Two supply Brescia, two interconnect with the Santa Massenza station* on the system of Società Idroelettrica Sarca Molveno, one will receive the supply from Boazzo, and provision is made for three further lines. On the 220 kV system, provision is made for solidly earthing both of the transformer neutrals, but only one of them is usually earthed in order to keep any possible fault currents within reasonable limits.

As will be noted from the diagram, the 220 kV system is kept completely separate from the other station supplies. There is no connection either with the auxiliary services or with the 9·2 MW Francis set.

The set just mentioned is connected to a C.E.M. 12 MVA three-winding transformer which provides secondary voltages of 15 and 130 kV. On the 130 kV side three lines will eventually be taken to Storo, Campo, and La Rocca. On the 15 kV side four feeders are supplied serving irrigation and reclamation schemes, and a connection is available to assist the station auxiliary services.

The 1,250 kW auxiliary set feeds a single busbar at generator voltage (3,150 V), and the auxiliaries for each of the three main generating sets (including the Francis) are supplied at 380 V through a separate 150 kVA air-cooled transformer. A fourth air-cooled transformer, of 500 kVA capacity, feeds a four-wire system supplying the station general services and battery charging at 380 V and lighting at 220 V.

Various connections have been made to safeguard the auxiliary supplies. We have already mentioned the connection from the 15 kV system supplied from the Francis set, and this is stepped down to 3,150 V through a 750 kVA transformer. A second busbar at 15 kV will eventually serve two feeders connecting to the auxiliary supplies at Boazzo and Storo and a third feeder supplying the equipment at Ponte Murandin. This busbar will be connected to the 3,150 V station auxiliary bar through a second 750 kVA transformer, and a cross connection on the high-voltage side of

* We hope to describe this station, which is the largest in Italy, in a future issue.

these two transformers will connect it to the 15 kV supply from the Francis set. Thus Cimego can either supply or receive auxiliary power to or from Boazzo or Storo in case of need. Further, the auxiliary set is capable of supplying all the Cimego auxiliaries, but alternatively the lighting and general services can be run independently from the 15 kV Francis supply; a busbar on the 380 V side also enables this supply to take over the auxiliaries of any main set in the event, for instance, of a breakdown of the transformer concerned. Finally, should the auxiliary set be out of operation, all auxiliaries can be run by the supply from the Francis set, assisted if necessary by supplies from the other stations.

Control Room

The control room (Fig. 24) occupies the top floor of a four-storey annexe built out centrally from the rear wall of the station, the associated cables, relays and ancillary equipment being accommodated on the lower floors. It occupies a strategic position, as the windows along one side overlook the machine hall

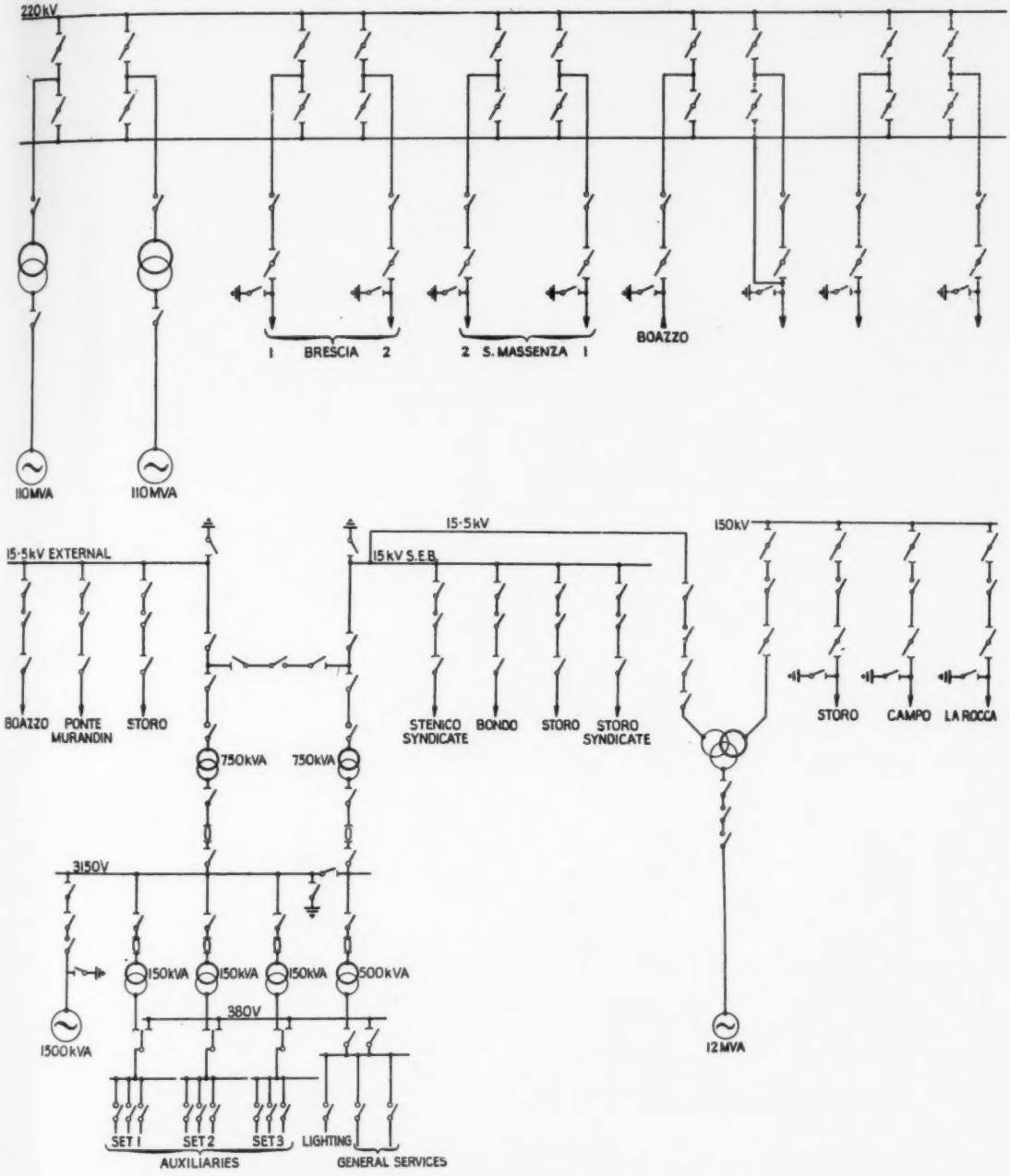


Fig. 23. Cimego circuit diagram, showing the 220, 150, 15·5 kV and auxiliary systems

Fig. 24. View of Cimego control room taken during completion

and those on the other overlook the switchyard. Four control boards are disposed round the room in the form of a hollow square. Facing the machine hall is the board controlling the generating sets, having panels on the left for the two main sets and on the right for the Francis and the auxiliary sets. All the machines can be remote started from this board, and synchronising is effected automatically. Facing the switchyard is the board controlling the 220 kV and 130 kV systems; to the left is a vertical board handling the 15 kV distribution system and the station batteries; and to the right is a board carrying recording instruments. The switchboard for the auxiliary services at 3,150, 380 and 220 V is located on the ground floor of the annexe, near to the room where the air-cooled transformers are located.

Conclusion

When we visited the Alto Chiese development, the Malga Bissina dam was in course of construction and the Cimego station, although in operation, was still in the course of receiving its final touches. Nevertheless we saw enough to impress us not only with the boldness of the engineering conception but also with the sense of artistry that the Italian engineer succeeds so happily in applying to his constructions.

In publishing this article we have great pleasure in acknowledging our indebtedness to Dr. Claudio Marcello, Manager of the Hydro-Electric Construction Department of Società Edison, who accorded us the fullest facilities to inspect the development and placed all needful information, drawings and photographs at our disposal. Amongst Dr. Marcello's immediate staff we would particularly like to acknowledge the services rendered to us by Ing. L. Noé and Ing. G. C. Adani.

The following is a list of the principal contractors concerned in the construction of Cimego station:—

Civil Engineering and Building
S.A.L.C.I.—Soc. per Azioni Lavori e Costruzioni Idrauliche—Milano.

Cranes
Gentili Brighi, Milano.

Penstocks
Officine Bossi—Milano, for the Murandin supply.
Acciaieria e Tubificio di Brescia, for the Boazzo supply.

Valves
For penstocks: A.T.B.—Brescia.
For the machines: Riva—Milano.

Turbines
Riva Costruzioni Meccaniche—Milano.

Pumps
A. Pellizzari & Figli, S.p.A., Arzignano.

Alternators
Main: Ercole Marelli & Co., S.p.A., Milano.
Francis and Auxiliary Sets: Breda Elettromeccanica e Locomotive S.p.A., Milano.

Transformers
13·8/220 kV: Ercole Marelli & Co. S.p.A., Milano.
10/15/130 kV: C.E.M.—Milano.
15/3 kV: „ „
3,150/380 V: „ „

Transformer Fire Protection
Mather & Platt Ltd., Manchester.

Circuit Breakers and H.V. Instrument Transformers
Scarpa & Magnano, Savona.

Disconnecting Switches
Bagnagatti—Milano.

Control Room
S.C.E.I., Novara.

Auxiliary Switchboard
Magrini S.p.A., Bergamo.

Transmission Towers
Soc. An. Elettrificazione S.p.A., Milano.

Line Fittings
A. Salvi S.A.—Milano.

Insulators
Richard Ginori—Milano.

Instrument Transformers
C.G.S.—Monza.

220 kV Bushing Insulators of Power Transformers
Scarpa & Magnano—Savona.

Segregated Bars
Bagnagatti—Milano.

Cavitation in Turbomachines

PART ONE

A selective survey of information available shows that in the absence of a definite cavitation pressure, scale effects are possible, that damage location has been alternatively related to two different phases of the cavitation cycle, and that cavitation pitting may occur under operating conditions of unimpaired machine performance

By F. A. L. WINTERNITZ, Dipl. Ing. E.T.H., V.D.I.*

CAVITATION first emerged at the turn of the century as the limiting factor in the design of hydraulic equipment. With the advent of high-speed ship propulsion and high-head turbine installations cavitation caused serious loss of efficiency and severe material damage, which brought home the need for effective design measures. The efforts since those early days to deal with the cavitation problem have been unrelenting. The results, however, have barely kept pace with requirements conditioned by intensive competition on an international market. The trend of development was to cut costs by reducing overall size and weight. In consequence the large water turbines and pumps of today are designed for high specific speeds and high specific loads, which has drastically enhanced the dangers of cavitation. Efficiencies are high, and progress by extension of their useful range is often more desirable than further efficiency gain. Here, once more, cavitation obstructs advance.

The need for research is more pressing today than when the first setbacks due to cavitation were experienced. Then the field was new and offered quick returns for moderate investments. Since then, experimental and field experience have led to such substantial improvements that further advance will necessitate much greater efforts. The very significant increase over the last fifteen years of capital investment in modern cavitation research facilities is evidence that the greater outlay is deemed well justified.

Research concerned with cavitation in turbomachines has followed three main lines:

1. Scale-model investigations have been used to establish cavitation characteristics in the operating range;
2. The resistance characteristics, under cavitation attack, of materials suitable for use in turbomachines were studied; and
3. The physical phenomenon of cavitation, the origin, growth and collapse of bubbles as well as the related mechanism of cavitation damage have been investigated.

The published material from a large number of disjointed investigations is voluminous. This paper has been written in the belief that there is a need for a summary of present-day knowledge in the field, and in the hope that some of the information, which is from sources not readily available, will be of value to the designer of hydraulic turbomachines.

* Fluid Mechanics Division, Mechanical Engineering Research Laboratory, East Kilbride, Glasgow.

Physical Aspects of Cavitation

Cavitation is a dynamic phenomenon, controlled by the hydrodynamics of the flow and the physical properties of the liquid, which commences once the pressure is reduced to vapour or saturation pressure. Apart from a reduction in ambient system pressure, or increase in the temperature-dependent vapour pressure, any localised dynamic pressure reduction may give rise to cavity formation. Fig. 1 demonstrates the main types of local low-pressure zones in turbomachines.

Cavity formation and collapse are extremely rapid processes which have been investigated by the use of high-speed photography. Several different types exist. Typical are the individual bubble with a cycle of growth, travel and collapse, the steady-state cavities,[1,2] and in separated flow the cavities in vortex cores. The last two, although studied on simple axisymmetric bodies of revolution and sharp-edged discs, are of the type chiefly observed in turbomachines.

A mechanism of cavitation inception has been established for non-separating flow[3]. It is based on the growth of individual microscopic bubbles at fixed positions in the boundary layer of the test body whence, after attaining a certain size, they are transported into the "visible" cavitation inception zone.

Steady-state or attached-type cavities originate without preceding bubble growth once the ambient pressure is sufficiently low or the velocity adequately

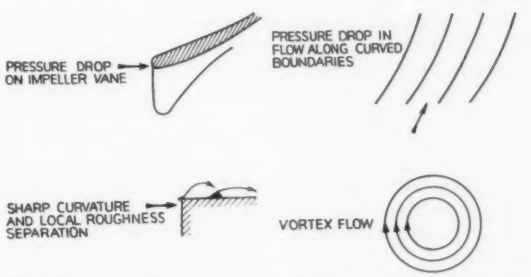

PRESSURE DROP ON IMPELLER VANE
PRESSURE DROP IN FLOW ALONG CURVED BOUNDARIES
SHARP CURVATURE AND LOCAL ROUGHNESS SEPARATION
VORTEX FLOW

Fig. 1. Some forms of local pressure reduction in turbomachines

high. The mechanism of their formation and collapse has been investigated by Knapp[2], who observed the existence of small bubbles which appear and grow at the upstream cavity end, travel along the interface, and implode in the downstream region of the cavity.

Vortex cavities are formed in the tip-clearance region of turbomachines, by eddy formation from obstacles or surface irregularities, through flow separation, jet mixing or in flow with a whirl component. Eddy cavities will collapse on entering a zone of higher pressure; cavities in attached vortices are broken up by the whipping vortex motion and collapse farther downstream.

Another form, "burbling" cavitation, is chiefly associated with propeller blades of short chord and appreciable thickness[4]. Here a multitude of individual bubbles travel at reduced velocities over the blade surface. They originate in the vapour-pressure range, apparently grow in that range while travelling downstream and distort and finally disappear before the point of full pressure recovery.

Ultimate success in controlling cavitation will depend on the ability to predict the form of cavitation which will occur in hydraulic equipment under specific conditions. This unfortunately depends on a number of factors difficult to define. It is, for instance, generally accepted today that cavitation originates from small gas or vapour nuclei which are stabilised on tiny solid particles in the liquid[5]. This nuclei concept has been advanced as a possible explanation for the occurrence of different cavitation forms on the same test body under nominally identical conditions[1]. From the available evidence it would appear that very low pressures are required to trigger off cavitation when relatively large nuclei are absent, and that inception at vapour pressure indicates their presence.

Untreated water containing nuclei can, however, sustain tensile stresses. Ziegler[6] in recent investigations on a large venturi-type test rig arrived at this conclusion, which was later confirmed by direct measurements with membrane-covered piezo tappings and air-filled pressure leads[3]. It is therefore possible that water may rupture at different pressures, depending on size and number of nuclei as well as the time of their passage through a low-pressure zone. Thus no definite cavitation pressure exists, and cavitation inception and subsequent development depend in a complex manner on molecular surface-contact phenomena. The evidence, however, is as yet not complete. Other variables are air or gas content effects on incipient as well as developed cavitation which have been investigated on stationary equipment and on propeller and turbomachine models.

Ziegler[6] found cavitation inception to be dependent on total air content and velocity of flow. Onset of cavitation occurred at vapour pressure for all velocities if the air content was high and air-bubble release preceded cavitation inception. At low air content, pressures below the vapour pressure were required to trigger off cavitation, the negative differences $p - p_v$ increasing with decreasing air content and increasing flow velocities. This suggests a further possible scale effect in cavitation tests on geometrically similar turbomachines under different test heads, since all velocities are proportional to the square root of the head utilised or generated by the machine.

Evidence of increased cavitation danger due to high dissolved air content was confirmed in recent water-tunnel experiments[7]. In Ref. 3 on the other hand no significant changes were noted in cavitation inception or hemispherical and ogival head forms for different air contents.

For turbomachines there is at present no definite experimental evidence to suggest air-content effects on either the initial phase or the developed stages of cavitation[8]. Medici† did, however, note that the state of repair of pump stuffing boxes was reflected in the pump characteristic, and attributed gradual breakaway to high and sudden breakdown to low air content. Marked air-content effects have been reported for the "burbling" cavitation of model ship propellers[4]. For "laminar" or sheet cavitation the performance was unimpaired. This latter form is similar to the type of cavitation observed on the runner blades of axial-flow turbomachines, and this suggests a measure of agreement between air-content test results in turbomachines and model propellers for fundamentally similar runners and types of cavitation.

Effects of boundary layer, of turbulence, surface tension and active materials[9] are too much still a matter of speculation to enter in this context.

The Mechanism of Cavitation Damage

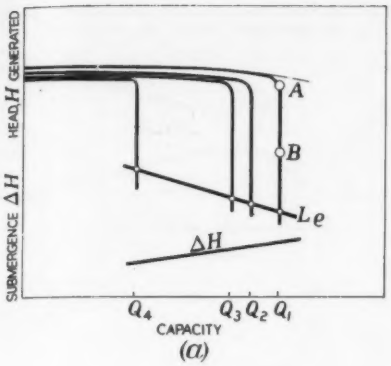
The practical importance of cavitation damage need not be stressed. There is at all times a requirement for improved protection of components exposed to cavitation attack, and thus a need for closer understanding of the nature of the attack. Economic considerations, for instance, frequently dictate operation of large water turbines close to or under tolerable conditions of cavitation. The costly repairs are accepted as a necessary evil. Also, extended requirements force turbine or pump operation outside the design range, resulting in pitted runners, casings and liners.

It is generally conceded today that cavitation is largely due to mechanical action. The collapse of an empty cavity in a liquid at rest was calculated by Lord Rayleigh[10]. Some of the conclusions have since been experimentally confirmed for a travelling cavity[1]. The calculated pressures resulting from the flow motion into the cavity are very high indeed, whilst thermodynamic treatment of the process still predicts pressures sufficiently large to cause fatigue failure of cast metals[11].

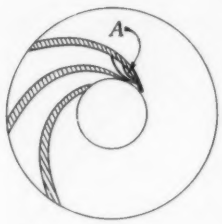
Since bubble collapse near a surface is distorted with corresponding reduction in pressure generated, it is questionable whether these are always adequate to cause damage to the material. This has given rise to a jet impact theory which assumes that small water droplets are accelerated by the collapse energy to very high velocities and impinge on the bounding surface. A fundamental similarity between drop impact and cavitation damage exists, and de Haller[12] has shown that repeated drop impacts of moderate magnitude can destroy material with very favourable resistance characteristics. Effects of corrosion in cavitation damage are still under investigation; different combinations of liquids and materials have been investigated[13] and the possibility of corrosion due to crystalline deformation in the material under cavitation attack has been considered[14].

For the fixed type of cavity, Knapp[2] has related the

† Medici, M. Personal communication; see also his "L'influenza delle condizioni reali di aspirazione su rendimento e prevalenza dell motopompe," *La irrigazione a pioggia*, 1955, No. 4, Figs. 7 and 8.



(a)



(b)

Fig. 2. (a) Characteristic of a single-stage condensate-extraction pump; (b) location of damage zone for operation at point A.

damage to individual bubble collapse of small cavities travelling along the cavity interface to implode at the rear of the cavity, where the zones of maximum damage are found. A cyclic process of cavity collapse and re-forming oscillates the stagnation point over an area. This concept links damage intensity to stagnation pressure and thus to free-stream velocity. Knapp also suggests that the total damage may prove independent of the length of the cavitation zone, whereas damage intensity may be found greatest for short cavities.

The writer believes that damage intensity is primarily controlled by the magnitude of the pressure available to collapse the cavities, which pressure also controls the extent of the cavitation zone. This may be shown by the example of the condensate extraction pump. The characteristic of such a pump, Fig. 2*a*, is peculiar in that flow control is automatically effected by runner cavitation, which is regulated by variations in submergence. Thus at one submergence the flow rate through the machine, and therefore the velocities, are essentially constant for a range of head requirements. If operated for a relatively short length of time in the region of the "break" *A*, substantial damage may occur, generally in the zone marked in Fig. 2*b*. However, for low head requirements, say point *B* in Fig. 2*a*, such pumps have been known to operate for many years without cavitation damage. In either case flow velocities through the pump are the same, but in case *A* the absolute pressure above vapour pressure is relatively high and causes short zones of cavitation. This is also in agreement with Nowotny's[15] magnetostriction results, where the influence of the available collapse pressure on damage was confirmed.

In a recent paper, Shal'nev[16] reports results which also seem in agreement with this pressure hypothesis. In his analysis, however, erosion intensity is linked to Reynolds number based on the diameter of the cylindrical test piece. A further remarkable conclusion of this paper is that, contrary to the conventional concept, the zones of maximum damage are claimed to occur at the point of bubble formation and breakaway from the surface of the test piece.

The pitting of materials under cavitation attack is essentially a fatiguing process under continuous impact blows which ultimately results in the chipping of the surface skin. On the roughened surface the process is accelerated due to stress concentrations. The importance of surface finish and selection of material for turbomachine components has therefore long been recognised[17].

The Cavitation Indices

The consequences of cavitation in hydraulic turbomachines—material damage, loss of performance, noise and severe vibrations—are clearly undesirable. Since it is uneconomic to design for cavitation-free performance, it is present-day practice to accept the occurrence of some local cavitation, and thus to tolerate moderate material damage and cavitation noise.

Loss of performance and severe vibrations, however, are unacceptable. In choosing the "safe" plant setting for his machine, the designer will consult the results of model tests. In these the cavitation indices of the model were determined as a function of operating conditions and presented in a form suitable for direct application to the full scale case. In all its stages cavitation is pressure controlled. It is therefore logical to use dimensionless pressure groups in classifying cavitation performance. The definition of these indices or parameters is essentially the same for all hydraulic equipment, stationary or in motion.

For steady flow in a stationary component, without gravity acceleration and negligible loss between two points, Bernoulli's theorem yields:

$$p_\infty + \frac{\rho V^2_\infty}{2} = p_m + \frac{\rho V^2_m}{2} \qquad \ldots (1)$$

and

$$2\frac{p_\infty - p_m}{\rho V^2_\infty} = \left(\frac{A_\infty}{A_m}\right)^2 - 1 \equiv K \qquad \ldots (1a)$$

where p_∞, p_m are the static, and $\frac{1}{2}\rho V^2_\infty$, $\frac{1}{2}\rho V^2_m$ the dynamic pressures at the two points and A_∞, A_m the cross-sectional areas.

Introducing the vapour pressure p_v into equation (1a)

$$2\frac{p_\infty - p_v}{\rho V^2_\infty} - 2\frac{p_m - p_v}{\rho V^2_\infty} = K \qquad \ldots (2)$$

where $2\frac{p_\infty - p_v}{\rho V^2_\infty} = \sigma$ is the cavitation parameter. Cavitation is taken to commence when $p_m = p_v$; therefore equation (2) becomes:

$$\sigma - \sigma_{crit.} = 2\frac{p_m - p_v}{\rho V^2_\infty} \qquad \ldots (3)$$

which is the pressure margin in per cent. of $\frac{1}{2}\rho V^2_\infty$ against a critical state of cavitation. The term "critical" is ambiguous. In tests on static equipment it generally denotes cavitation inception, determined visually and/or acoustically. In testing hydraulic turbomachines it refers to a developed state of cavitation affecting performance adversely. The change in per-

formance may be abrupt or gradual, depending on machine specific speed, and a further definition often used is a one per cent. drop in unit power or efficiency as denoting the critical condition[18].

Continental practice sometimes differentiates between $\sigma_{min.}$, relating to a minimum pressure at which cavitation in the machine is just avoided, σ_{normal} which contains a pressure reserve against the onset of cavitation, and $\sigma_{crit.}$ once pressures below the vapour pressure are reached. A reliable determination of $\sigma_{min.}$ would yield a criterion of safety against the occurrence of cavitation damage. In general, however, $\sigma_{crit.}$ as defined above is the value of the cavitation parameter determined in the experiment; modern hydraulic turbomachines operating at the accepted cavitation limits of present-day design practice cannot be expected to be free of local cavitation[19].

The choice of reference position p_∞ is arbitrary, as may be seen from Fig. *3a* and *b*.

An expression equivalent to equation (3) can be derived for a water turbine or pump:

$$\sigma - \sigma_{crit.} = \frac{p_{min.} - p_v}{\gamma H_{T,P}} \equiv \frac{M}{H_{T,P}} \qquad \ldots (3a)$$

which is the pressure reserve against occurrence of a critical cavitation condition at the point of lowest pressure in the machine, where $H_{T,P}$ = head utilised or generated, γ = weight per unit volume, M = margin of safety in feet (w.g.).

By means of an energy balance and use of the Thoma definition of the cavitation parameter[20], it can be shown that for a turbine:

$$\sigma_{crit.} = \frac{p_v}{\gamma H_T} + \lambda_1 K^2_{c_2} + \lambda_2 K^2_{w_2} \qquad \ldots (4a)$$

and for a pump

$$\sigma_{crit.} = \frac{p_v}{\gamma H_P} + \lambda^*_1 K^2_{c_1} + \lambda^*_2 K^2_{w_1} \qquad \ldots (4b)$$

where $K_c = C/\sqrt{2gH_{T,P}}$, $K_w = W/\sqrt{2gH_{T,P}}$ and the suffices of the absolute velocity C and relative velocity W refer in both cases to the runner suction side.

Since λ_1 and λ^*_1 are related to draft tube efficiency and suction-branch loss respectively, and λ_2, λ^*_2 are blade shape and position factors, the dynamic terms in the equations depend on discharge and suction-passage formation as well as runner design and speed.

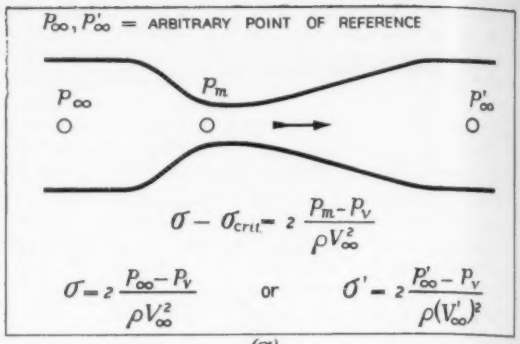
Approximate mean λ values for best efficiency as quoted by Pfleiderer[21]

Turbines: $\lambda_1 = 0{\cdot}7$ $\lambda_2 = 0{\cdot}16$
Pumps: $\lambda^*_1 = 1{\cdot}2$ $\lambda^*_2 = 0{\cdot}3$

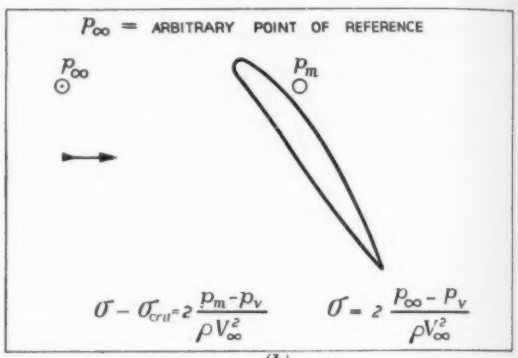
For critical conditions Gongwer[22] obtained from low-specific-speed pumps:

$\lambda^*_1 = 1{\cdot}4$ $\lambda^*_2 = 0{\cdot}085$

It is of interest that according to Krisam[23] $\lambda^*_2 = 0{\cdot}085$ is more in line with general experience than $\lambda^*_2 = 0{\cdot}3$. Wislicenus[19] points out that in the light of theory, values lower than 0·3 are fictitious insofar that they would require agreement between the relative flow direction and the direction of a very slender impeller blade to within a fraction of a degree. This is based on the contention that near "shock-free" entry is required for optimum cavitation performance, which in the same paper and elsewhere is shown to conflict with experience. Lower λ^*_2 values can be obtained for fairly large angles of attack and conventional blade contours. One possible explanation of the discrepancy between prediction and achievement is wrong assumptions regarding the inflow conditions at runner entry, whereby three-dimensional flow motion could possibly be responsible for the improvement of cavitation performance in a manner as yet not clearly understood[19].


P_∞, P'_∞ = ARBITRARY POINT OF REFERENCE
P_m
P_∞
P'_∞
$\sigma - \sigma_{crit} = 2\frac{P_m - P_v}{\rho V^2_\infty}$
$\sigma = 2\frac{P_\infty - P_v}{\rho V^2_\infty}$ or $\sigma' = 2\frac{P'_\infty - P_v}{\rho (V'_\infty)^2}$

(a)


P_∞ = ARBITRARY POINT OF REFERENCE
P_∞
P_m
$\sigma - \sigma_{crit} = 2\frac{P_m - P_v}{\rho V^2_\infty}$
$\sigma = 2\frac{P_\infty - P_v}{\rho V^2_\infty}$

(b)

Fig. 3. Definition of cavitation parameter for (a) venturi test section; (b) hydrofoil

Due to a reversal in the effect of friction, the same runner will perform more favourably as a turbine with regard to cavitation than when operating as a pump. The turbine coefficients appear to be favourable since draught tube efficiencies of 0·8 to 0·85 are realised, and λ_2 values greater than 0·2 are not infrequent. An approximate relation for the latter in the range $0{\cdot}5 < l/s < 2$ for axial flow machines is:

$$\lambda_2 \approx 0{\cdot}22 - 0{\cdot}25 \times s/l \qquad \ldots (5)$$

where s = spacing, l = chord.

Equations (4) and (4a) give the internal cavitation parameters of a turbine or pump with regard to a datum at the same static elevation as the velocity reference point in the runner eye. They facilitate comparison with aerodynamic results but are not particularly suited for the experimental determination of $\sigma_{crit.}$ Rewriting:

$$\sigma_{crit.} \cdot H_T = \frac{p_v}{\gamma} + \lambda_1 \frac{C^2_2}{2g} + \lambda_2 \frac{W^2_2}{2g} \equiv \Delta H_T \qquad \ldots (6a)$$

and

$$\sigma_{crit.} \cdot H_P = \frac{p_v}{\gamma} + \lambda^*_1 \frac{C^2_1}{2g} + \lambda^*_2 \frac{W^2_1}{2g} \equiv \Delta H \ldots (6b)$$

where ΔH is the minimum pressure head required at the runner eye to avoid the critical state of cavitation at the point of lowest pressure in the runner. For a

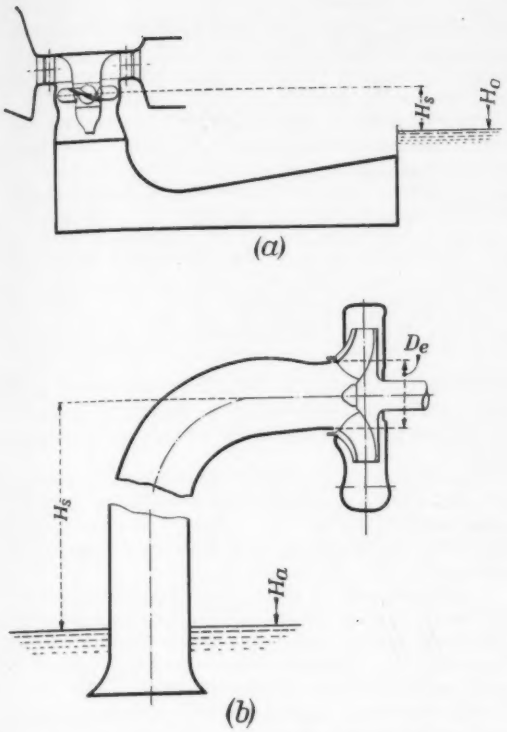


Fig. 4. Definition of static suction height

centrifugal pump with suction from a vacuum, equation (6b) represents the minimum submergence required.

The internal, that is the machine cavitation characteristics, are related to plant conditions by the Thoma definition of the cavitation parameter[20]

$$\sigma_{crit.} = \frac{H_a - H_{s\ max.}}{H_{T,P}} \qquad \ldots (7)$$

where H_a = barometric pressure height, $H_{s\ max.}$ = critical static suction height.

Equation (7) is, as part of the energy balance mentioned, equivalent to equations (4a) or (4b), but is more often used in a version containing the vapour pressure:

$$\sigma_{crit.} = \frac{H_a - H_{s\ max.} - \frac{p_v}{\gamma}}{H_{T,P}} \qquad \ldots (7a)$$

where $\frac{p_v}{\gamma}$ = vapour pressure of liquid. In consequence the terms $\frac{p_v}{\gamma H_{T,P}}$ and $\frac{p_v}{\gamma}$ will not appear in equations (4a) and (4b), and (6a) and (6b) respectively if the cavitation parameter is defined by equation (7a). This has the advantage of separating the turbomachine and the characteristics of the plant in the σ definitions.

The safe setting of the machine in the plant may be determined by means of equation (7a) from:

$$H^*_{s\ max.} = H_a - \Delta H_{T,P} - M \qquad \ldots (8)$$

for a machine with vertical shaft, Fig. 4*a*, and

$$H^*_{s\ max.} = H_a - \Delta H_{T,P} - \frac{D_e}{2} - M \qquad \ldots (8a)$$

for a unit with shaft horizontally arranged, Fig. 4*b*. $H^*_{s\ max.}$ = maximum permissible static runner elevation above lowest tailrace or sump level. In the case of a vertical Francis unit $H_{s\ max.}$ refers to the runner discharge plane.

The term M is a safety margin often determined by economic considerations. Ideally, the plant setting must ensure for all operating conditions suction heights H_s below the permissible one. The plant sigma will then always be larger than the $\sigma_{crit.}$ of the unit. Once the setting of a large hydraulic turbomachine has been chosen, the permissible discharge or power output will be controlled by the plant sigma, which changes with head and tailrace variations, with barometric pressure and with seasonal temperature changes.

Evidently measures concerned with the improvement of cavitation features of a turbomachine must be directed towards lowering ΔH, that is reduction of the dynamic terms in equations (6a) and (6b) within the given range of operation.

Between the cavitation index σ and the specific

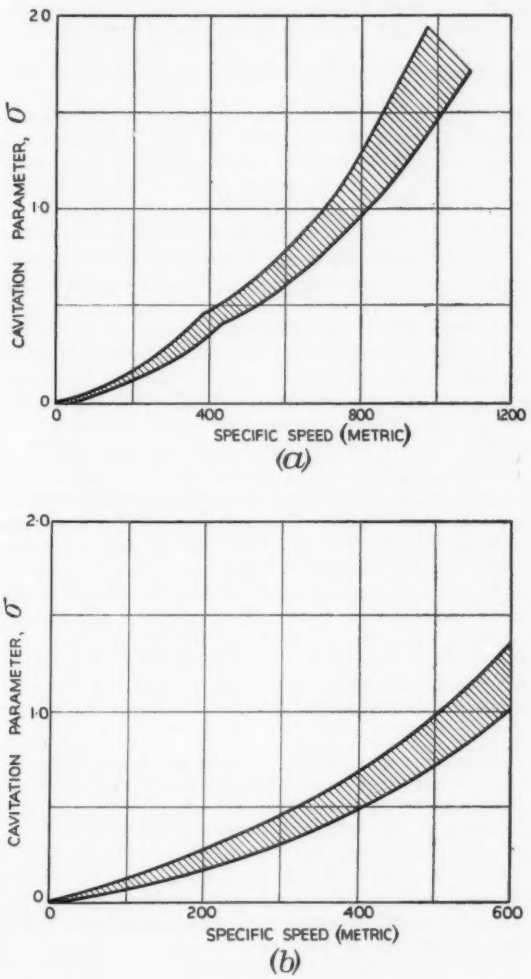


Fig. 5. Approximate critical cavitation parameter for (a) Francis and Kaplan turbines; (b) centrifugal pumps

speed N_s there exists a well-known relationship:

$$\sigma = \text{const.} \times N_s^{4/3} \qquad \ldots (9)$$

where $N_s = \frac{N\sqrt{Q}}{H^{3/4}}$ is the rotational speed of a runner of such dimensions that unit flow will result under unit head. Fig. 5*a* and *b* give practical values for modern turbines and pumps.

Another cavitation parameter of the same structure as specific speed was derived by Pfleiderer[21]. Expanding the dynamic terms in equation (6b) for flow with and without a whirl component, and introducing the angle between the relative velocity at inlet and the rotational direction as well as the runner eye diameter, the following equation was obtained:

$$S^* = \frac{Q}{k\Delta H^{3/2}}\left(\frac{N}{100}\right)^2 \qquad \ldots (10)$$

where Q = flow rate in cu. m. per sec., $\Delta H = \sigma H_{T,\,P}$ in metres, N = machine speed r.p.m. and $k = 1 - \frac{D_h}{D_e}$, where D_h and D_e are hub and eye diameter respectively. S^* is a function of the angle of attack, defined at the runner-eye diameter, and of the λ factors only, and will be constant as long as their value does not change. Any comparison based on S^* must refer to conditions of optimum efficiency, and will be affected by roughness, size and Reynolds number.

The important difference between σ and S^* is that S^* has been confirmed experimentally to be practically independent of N_s and of the pressure head $H_{T,\,P}$, in the range of low and medium specific speeds. This makes the suction specific speed an adequate criterion for comparison of suction performance in machines with similar inlet passages but of different specific speed. The overall runner shape does not enter.

From the hydrodynamic considerations alone it would thus follow that a geometrically similar model should correctly reproduce the suction performance of the full-scale machine, irrespective of the absolute magnitude of the test head used in the model experiments. By the same token it must, however, be remembered that S^* was derived by use of the continuity equation, which is valid only as long as the flow is uniform and free of cavities, and further that the underlying assumption of entry conditions unaffected by runner discharge performance will not be adequately fulfilled in hydraulic turbomachines of high specific speed.

The parameter S^* was termed the suction specific speed by Wislicenus and collaborators[24] who derived, from similarity considerations alone, for use with centrifugal pumps, the parameter:

$$S = \frac{N\sqrt{Q}}{\Delta H^{3/4}} \equiv 100\sqrt{S^* k} \qquad \ldots (11)$$

Suction specific speed and machine specific speed are related by:

$$S = \frac{N_s}{\sigma^{3/4}} \qquad (12)$$

Equation 12 may be used to calculate S values from available cavitation test data. This has been done for water turbines,[21] and results in metric units and for maximum flow through the machines are reprinted in Table I.

TABLE I

Francis turbines							
N_s =	50	80	100	200	300	400	500
S^* =	4·7	6·35	8·18	10·0	9·18	7·02	5·9

Kaplan turbines				
N_s =	450-500	550-600	650-700	700-800
S^* =	9·1-9·4	7·35-8·5	6·1-7·03	5·1-6·6

$$S^* = \left(\frac{1}{3{\cdot}65}\right) \cdot 10^{-4} \cdot \frac{1}{k} \cdot \frac{1}{\eta_t} \cdot \frac{N_s^2}{\sigma^{5/2}}$$

For comparison, a mean value cited for pumps is $S^* = 2{\cdot}6$ (this corresponds to $S \sim 7{,}600$ (ft., gal. per min.) which falls into the range $7{,}000 < S < 10{,}000$ of Ref. 25). The implications drawn from these numerical results are discussed in Ref. 21.

The usefulness of the suction specific speed lies in the possibility of comparing suction performance in turbomachines of different specific speed by relating cavitation performance solely to runner suction and suction or discharge branch shape.

This has been used to breed optimum pump-suction performance. However, since the experimental determination of S^* is obtained from equation (10) and $\Delta H = \sigma H$ must first be known, it is much simpler to use σ and equations (8) and (8a) for individual model tests. Where a series of model runners covering the N_s field is available, the use of $\sigma_{crit.}$ curves of the runner selected for a given application will be preferred when determining the safe setting and operating range of the machine. The design of a new runner, however, may be guided by an analysis of suction specific-speed values, registering the effects of modifications on the runner and machine suction side. S^* is as yet rarely used in water-turbine design although widely accepted by pump manufacturers, in particular in Britain and America.

Thermal equilibrium considerations for the case of given geometric and kinematic similarity lead Stahl and Stepanoff[26] to suggest a thermal cavitation parameter for those cases where the performance of pumps handling different liquids is of interest:

$$B = \frac{v_v}{v_L} \cdot \frac{\Delta h_l}{L} \equiv \frac{V_v}{V_L} \qquad \ldots (13)$$

where v_v, v_L are the specific volumes of vapour and liquid, Δh_l is enthalpy in thermal units per unit weight of liquid, L represents the latent heat of vaporisation, and V_v, V_L are the respective vapour and liquid volumes.

The absolute value of B, which is applied as a relative criterion, is not considered, and it is argued that the manner of its use justifies the inaccurate assumption underlying the heat-balance equation, that infinite time is available to restore the thermal equilibrium. Once the cavitation characteristics of a pump handling, for example, water at different temperatures are known, its performance when pumping other liquids can be predicted by determining B and selecting that

characteristic performance curve from the tests with water which, for the same assumed drop in pump inlet pressure, gave the same or nearly the same B value.

Other cavitation indices, derived from similarity considerations, have been found useful in presenting experimental data[22, 25]. For serial-type machines, semi-empirical relations exist as the result of statistical evaluation of a large number of tests results[27-29].

The indices discussed so far cannot securely safeguard a turbomachine against cavitation damage. They were derived under assumptions for which the lowest pressure will always occur on a bounding surface. By and large this remains true for the flow in turbomachines. The action of the boundary layer may however give rise to vortex and eddy formation, with low-pressure core cavitation. Ideally the core pressure is related to circulation intensity by:

$$p_c = p_\infty - \frac{\rho \Gamma_o^2}{4\pi^2 r_i^2} \qquad \ldots (13)$$

where Γ_o = circulation, r_1 = radius of vortex core. Equation (13) describes tip vortex condition approximately as outlined by Ackeret[30]. Thus material damage may occur in turbomachines operating under safe conditions according to equations (8) and (8a). Shal'nev[31] points to the danger of tip clearance and roughness cavitation in this respect. The same author and collaborators have carried out systematic investigations in hydraulic power plants by use of scale photography[32, 33]. Guided by the criterion of cavitation damage, they derived further cavitation indices defining clearance and roughness cavitation. These can become critical before profile cavitation sets in, that is at a higher σ than predicted by Fig. 5*a* and *b*. Numerical coefficient values for calculations are taken from experimental data[34, 35].

(To be continued)

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Norwegian-Russian River Agreement

The Norwegian-Soviet Expert Commission for the utilisation of the water power in the Pasvik River, the border river between Norway and Russia in the Far North, has now agreed and submitted a draft agreement between Norway and the Soviet Union. According to the agreement, the Soviet Union will be given the right to utilise the water power in the lower and upper parts of the river, while Norway will be given the rights in the middle part.

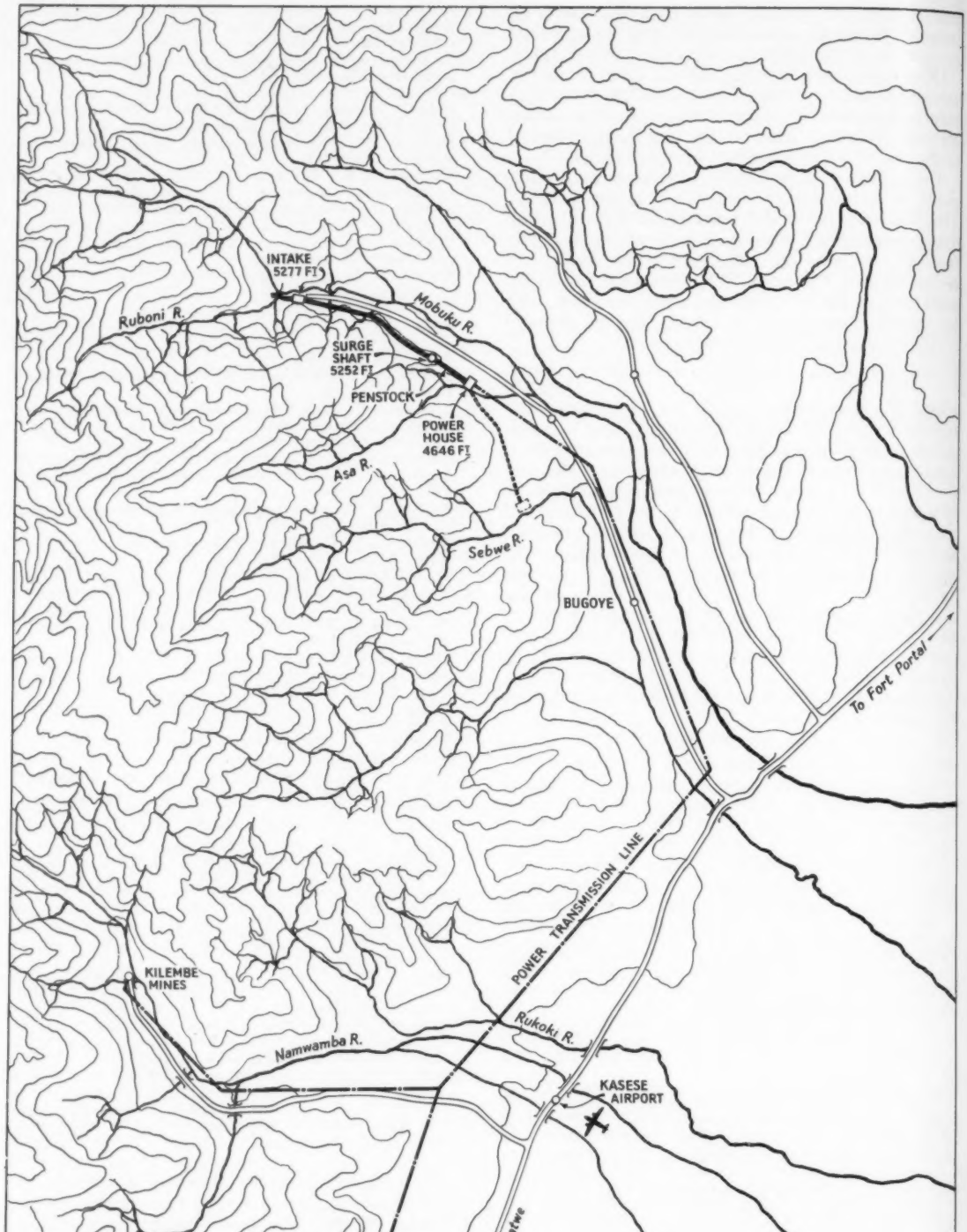


Fig. 1. Sketch map of Mobuku and Namwamba valleys **KILEMBE POWER PLANT**

Kilembe Power Plant

A 9,000 h.p. hydro-electric development in Uganda, for Kilembe Mines Limited, is described by F. J. Bartholomew, F.A.I.E.E., and M. M. de Leva, A.S.C.E., A.I.E.E.

DURING the years 1946 to 1947, Frobisher Limited, associated with Ventures, two well-known Canadian mining development companies, undertook the examination and exploration of a copper-cobalt prospect in the southern foothills of the Ruwenzori Mountains in Uganda. The site is a few miles north of the Equator, and the altitude is 4,500 ft. As part of this work, Mr. F. J. Bartholomew was invited to investigate the possibility of generating hydro-electric power on one or other of the several streams draining the south-eastern section of the Ruwenzori watershed. The mountains here rise to a height of 16,700 ft., and perpetual snow fields are to be found above the 14,000 ft. level, with occasional snow reaching down as low as 10,000 to 11,000 ft.

At the end of some two years of observations it became apparent that the more northerly stream, the Mobuku, was the obvious first choice for power development. Its run off area of 72 sq. miles was the largest, and included the greater part of the snow fields; it also had extensive marshes, at an elevation of between 9,000 and 10,000 ft., which act like a sponge and tend, together with the snow, to maintain an extraordinarily even flow, considering the smallness of the total watershed and the mountainous country in which the valley lies.

At that time there was a complete absence of accurate precipitation records and hydrological information. There were no reliable maps of the area, and even the geographical relationship of the Mobuku to the mine was only known from mileage readings by automobile to the nearest point of road access. From the end of the road, trails had to be cut through dense elephant grass growing to a height of 8 to 10 ft. to the stream banks.

A programme of rainfall and flow measurements was started in 1947, and it was found that between that date and 1953 water measurements gave highs of between 3,000 to 4,000 cusecs, lasting a few hours, and lows of 120 to 140 cusecs, that persisted even a month or so during dry years. A study of these records made it appear that in most years the minimum flow

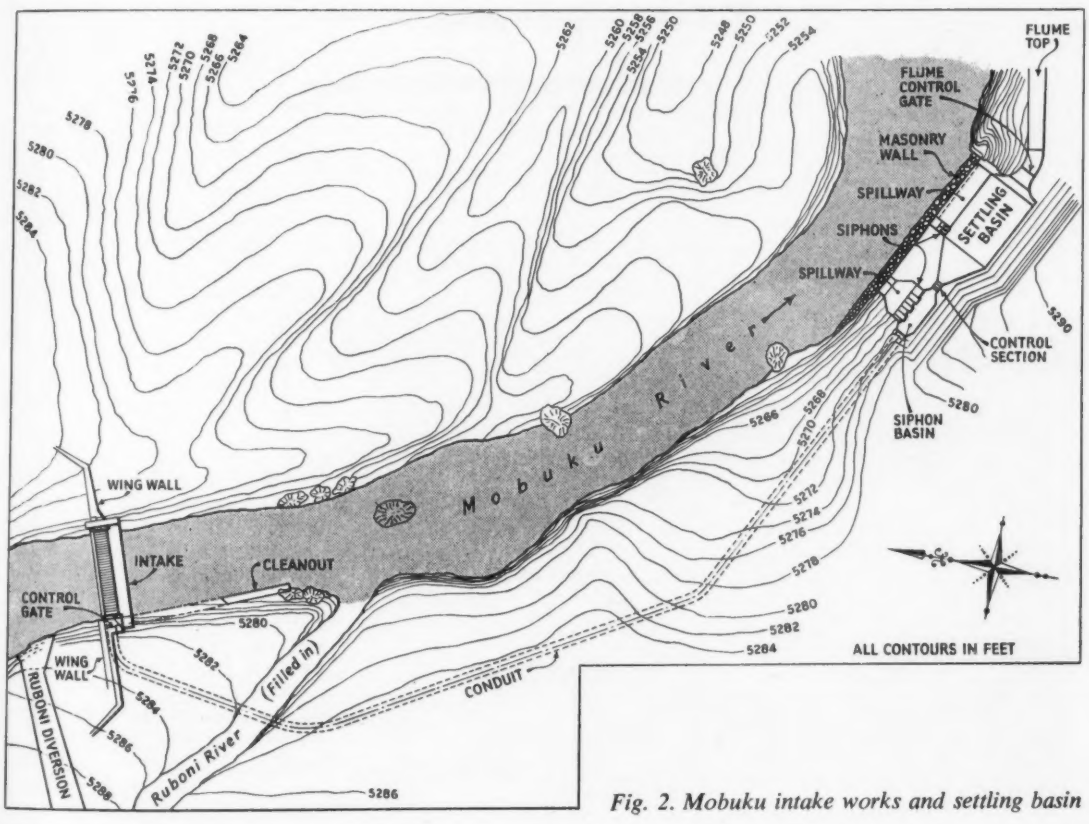


Fig. 2. Mobuku intake works and settling basin

could be expected to be not less than 140 cusecs for eleven months.

As in most tropical areas, there are two rainy seasons, one from March to May and the other from September to November. Whilst the wet seasons provide much the heavier precipitation, rainfall is fairly well distributed throughout other months, with the exceptions of January and sometimes June and early July.

The precipitation at 4,000 ft. altitude is approximately 45 in. per annum, but further up the valleys it is greater, and whilst no records are available at 9,000 to 10,000 ft., it must be well in excess of 100 in. The mountain tops are shrouded in clouds for at least 90% of the year, and it is an occasion when one can see the snow fields from the plains.

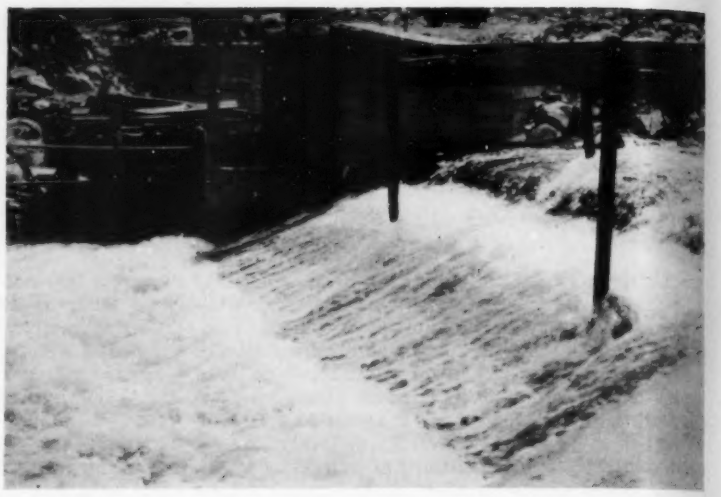


Fig. 3. View of intake works

The Mobuku itself is a stream 50 to 80 ft. wide and lies in a valley that, at the point selected for the present intake, broadens out to a width of half to one and a half miles. The stream banks vary in height from 4 to 10 ft., and examinations of the valley bottom showed that it had in previous times cut other courses. The valley bed itself comprises boulders and gravels throughout its entire extent, probably to depths of up to 200 ft.

Further investigations during the following years brought to light the fact that in 1930, and probably some twenty-five to thirty years previously, there had been tremendous floods with flows of 15,000 to 20,000 cusecs, which lasted several days. Apparently these

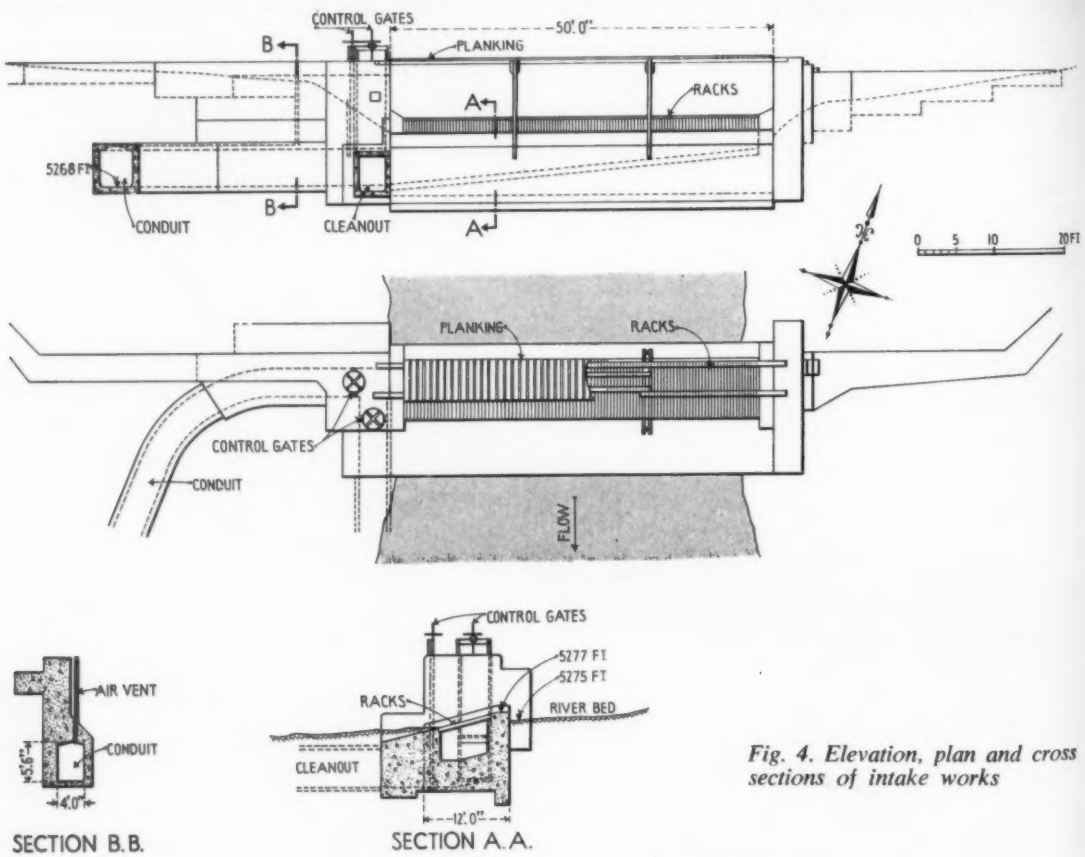


Fig. 4. Elevation, plan and cross sections of intake works

Fig. 5. Settling chamber

started with an onrush that was so sudden that Africans in the valley had been carried down and drowned, and even elephants had been washed away by the tremendous body of water.

By 1953 the mine development had reached a stage where some ten to twelve million tons of ore had been proven and indicated. A decision was reached at that time to start operations, and it was determined that some 6,000 h.p. would be needed, allowing for transmission and transformation losses. In view of probable expansion, it was decided to design the intake and waterways to the forebay for an ultimate capacity of 9,000 h.p., and to build a first power plant with 6,500 h.p.

A two-unit plant with a single penstock was decided upon and the layout designed to permit the addition of two further sets with a second penstock, thus providing a spare unit in the final development. Construction along these lines was finally authorised during the summer of 1954, and work started in November of the same year.

It was immediately evident that any form of dam was quite impracticable. No rock foundations were available, and any dam structure raised materially above the river bed would fill up in the course of a year or two, thus presenting a severe obstacle to the movement of heavy boulders; also any normal type of weir would tend to cause a diversion of the stream to the north and render it difficult to restore it to the present channel. An examination of the site showed that diversions of this nature must have occurred at some time in the past.

It was decided, therefore, to construct an absorb-

Fig. 6. Mobuku below settling chamber

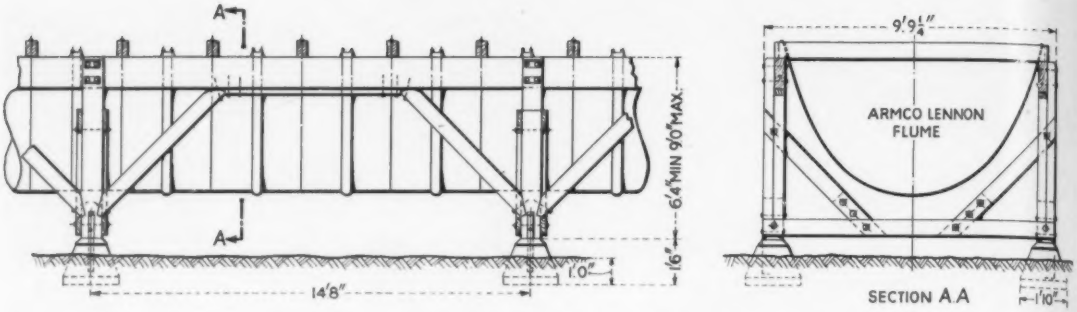


Fig. 7. Details of Lennon flume

tion intake of a type that has been used in the mountains of northern and central Italy under similar conditions, which has the advantage of not changing the contour of the stream bed. This is done by a concrete trench intake that crosses the stream from bank to bank, the upstream wall of the trench being approximately 1 ft. above the downstream one that is prolonged in an apron flush with the original stream bed.

The trench was made 7 ft. wide and 3 to 7 ft. deep. Extensive wing walls were constructed into the banks on both sides to regulate the flow of the water over the intake. The deep end of the trench was connected to a concrete duct with a sluice gate to control the flow of water. A smaller duct at a slightly lower level, also with a sluice gate, directed downstream, serves to carry away small stones and sand which might collect in the trench.

Heavy steel bars at $\frac{3}{4}$ in. spacing cover the trench at stream level from wall to wall, acting as racks to keep out stones, boulders, branches, etc. These bars have been proportioned to carry the heaviest boulders likely to be washed down in conditions of extreme flood.

The south bank concrete work and the wing dam have been carried back from the river at a height that will permit a stream flow in excess of 20,000 cusecs without damage to the intake works. Provision has also been made to divert a substantial proportion of the water flow under this extreme condition to an old channel above the intake and to the north.

The necessity of taking into consideration such rare but not impossible extreme floods constituted perhaps the major problem of the project, but with the design adopted there is confidence that, apart from minor trouble due to the cutting of secondary channels and the deposit of gravel at various points that may take a week or so to eliminate, there should not be any real damage of a major nature.

The concrete duct from the intake is buried under several feet of replaced overburden, and protected by a heavy embankment of boulders and gravel to a height of 8 or 9 ft. for a distance of some 300 ft.

The Ruboni, a small stream with very constant flow that originally cut across the path of this duct downstream from the present intake, has been diverted upstream above the protective embankment so that its water is added to the useful flow of the Mobuku.

The concrete duct at some 500 ft. from the intake emerges from the ground into a control chamber and a settling basin. At the end of this basin the water is directed into a metal flume, of semi-circular cross-section approximately 8 ft. wide, that has a nominal flow capacity of 155 cusecs with a freeboard of 6 in. A sluice gate controls the flow of water from the settling chamber into the flume.

One of the requirements of this particular design of flume is that the maximum water level must be carefully controlled, as any excess of flow will tend to spill over the sides, especially if there is any wave action due to inequalities in construction, and this overflow, if unchecked, could damage the whole hill bench. To meet this condition it was decided to control the height of the water in the settling chamber to within 1 in., irrespective of the amount of water coming out of the duct, this being influenced by the variations in river flow.

To effect a first rough automatic control the concrete duct was designed to carry the full quantity of water approximately 85% full; any flow in excess of this amount will fill the duct, thereby creating addi-

Fig. 8. Flume in course of assembly

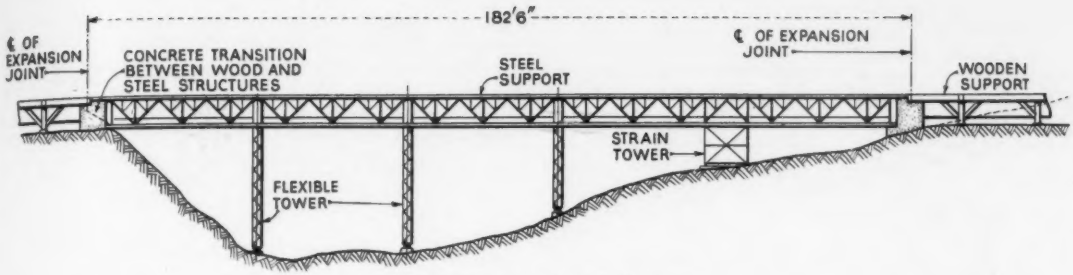


Fig. 9. Steel trestle for flume

tional friction due to the increased wetted perimeter.

Even so, calculations showed that under extreme conditions, some 250 to 300 cusecs might still arrive at the settling chamber. It was, therefore, decided to end the duct with a control section containing three syphon spillways capable of discharging a total of 150 cusecs back into the stream, with a rise in water level of 2·5 in. This control chamber is connected to the settling basin by a restricted passage with adjustable baffles that are set so as to create the required difference in head between the syphons and the settling chamber to be controlled. The final adjustment in the water level occurs in the settling chamber itself by means of a 40 ft. spillway with an adjustable steel lip and two small syphon spills having a capacity of 12½ cusecs each under a variation in water level of ¾ in.

A wash-out gate in the settling chamber serves to discharge such pebbles and sands as enter the duct despite the wash-out sluice at the intake.

Figs. 2 to 4 show the construction of the intake with its associated works, including the settling chamber.

From the settling basin to the forebay there is a flume 11,628 ft. long that includes a 506 ft. tunnel section to avoid a long detour around a projecting spur in the hillside. This flume is the Lennon type, manufactured by the Armco Company of Middletown, Ohio, and consists of a series of individual sheets 3 ft. long, each having matched overlapping grooved ends which were treated with a bitumen compound during erection, and then compressed together by a half-round steel rod, pressed into position by steel plates held down by tie-rods. This construction, Fig. 7, gives a virtually leak-proof construction, with sufficient elasticity to allow for thermal contractions and expansion. The flume grade is ·15%, and the maximum deflection at curves 4° over a length of 14 ft. 6 in.

East African hardwoods were used for the timber structures supporting the flume. Considerable study was given to the choice of the most suitable among the various local timbers, and preference was given to three, namely, Whitestar Apple, Dahoma, and Malamancobi. Further study was also given to the best wood preservative impregnation to ensure a reasonably long life to these structures. Although the flume lies at an elevation of above 5,000 ft., which is above that at which ants constitute a serious problem, it is felt that fungus attack will still exist, even if not in such a serious form as at lower elevations.

Of course, the best method of preservation would have been pressure impregnation, but no adequate plant was available locally, and a special installation would have proved too costly. As a second best choice it was decided to adopt hot and cold baths, using a mixture of creosote and a solution of 5% pentachlorophenol in diesel oil. In this method the individual members were first cut and dapped to size, and then immersed first in a hot solution for a sufficiently long time to permit the expulsion of all surface moisture and then passed into a cold solution intended to press in the superficial absorption of the first bath. Penetration proved to vary from ½ to 1 in., despite the compactness and the hardness of the fibres.

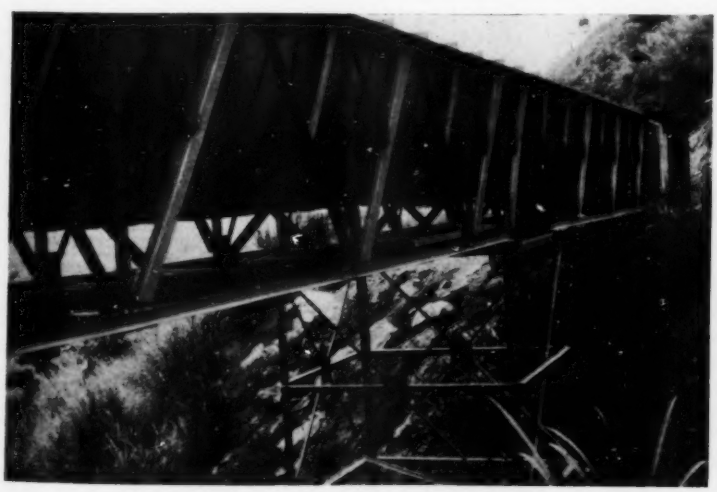


Fig. 10. View of trestle carrying flume

Consideration was given to the use of steel structures for supporting the flume, but a cost analysis indicated that the expense would have been substantially greater, and confidence is felt that with due maintenance, a twenty-year life should be obtained.

To accommodate the flume it has been necessary to cut along the face of the hillside that has a slope of the order of 50°. The surface of this hillside is mostly laterite covered by a dense growth of elephant grass. Several small streams flow off the mountains, creating gulleys that had to be bridged to avoid sharp bends

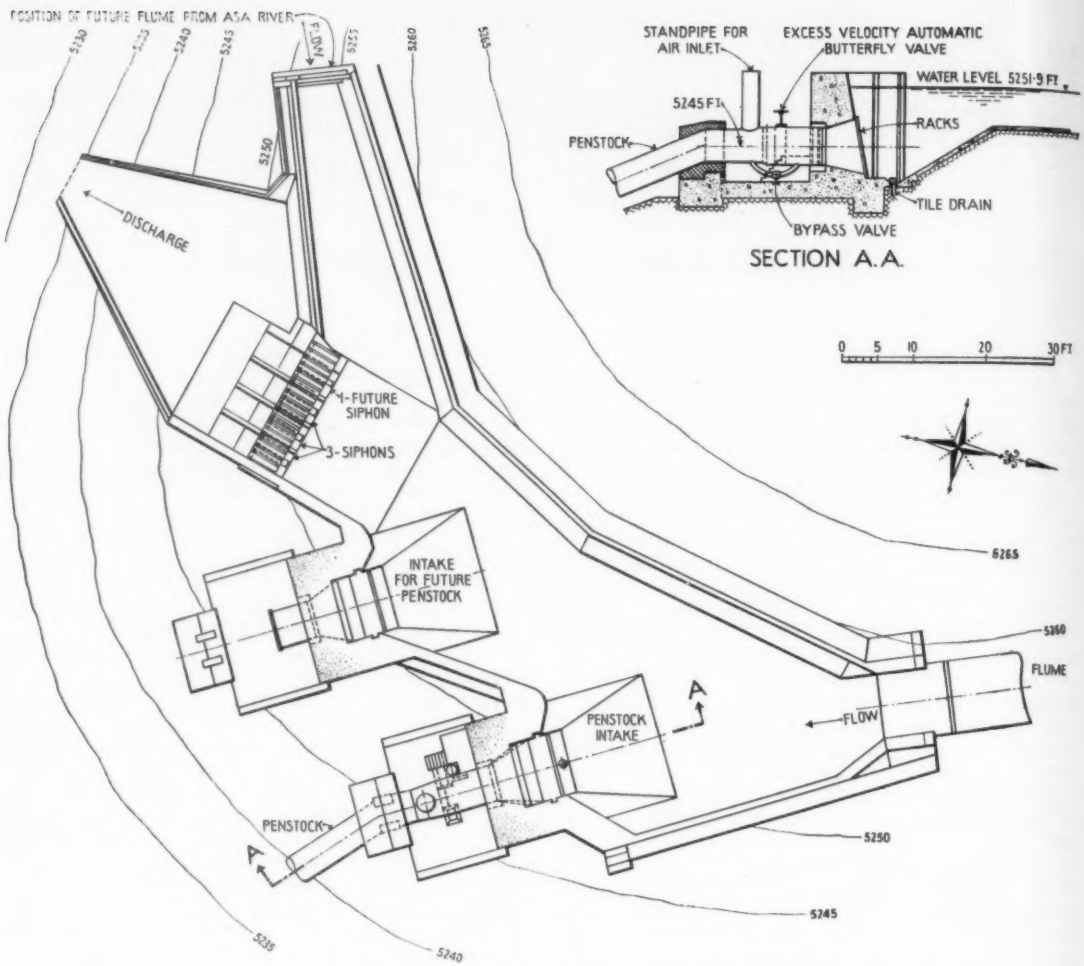


Fig. 11. Arrangement of concrete forebay

or excessive excavation. Steel was used for the trestling of some of the bridges over the gulleys (Fig. 9).

Concrete footings were provided to support the vertical posts of the flume structure. The depth and size of these were adjusted to suit the nature of the bench; standard footings were 30 in. in good ground, but it was necessary to sink quite a number to a depth of 5 to 6 ft.

For the greater length, the hillside above the cut is self-supporting on slopes of 40° to 50°, but one major slide and some smaller ones have occurred, and a year or two will pass before conditions become stabilised. Elephant grass planted on the steep slopes quickly develops a good growth, and the thickly matted roots that go down to a depth of a foot or two will prevent surface erosion. It is not, however, intended to permit elephant grass to grow on the bench, as it would do in three to six months if left to itself. Instead, a low growing thick tough grass called "kukui" has been planted on the berm. This grass spreads and grows very quickly, keeping away the elephant grass, and it is expected that within a year it will have provided a tight matting that will prevent erosion without impeding access. Gumwood trees are also being planted in some locations as further protection.

About 3,000 ft. of the hillside turned out to have solid rock very close to the surface, and this has been responsible for a considerable increase in the estimated cost of the excavation.

The nominal quantity of water the flume was to deliver was set at 140 cusecs, but to allow for errors in construction, or for some slight settling of the foundations, the completed design of the waterworks was carried out for 155 cusecs.

The flume terminates in a concrete forebay, Fig. 11, which has been designed and built to allow the second penstock to be installed at a later date.

The forebay arrangement provides a continuing direct flow of the flume water into the penstock chambers, and has proved entirely satisfactory, with no eddying or tendency to create whirlpools sucking air, even with the penstock at full-flow capacity.

It was necessary to provide spilling facilities at the forebay to dispose of the water from the flume when the waterwheels rejected their load or shut down. This has been done by means of a 30 ft. spillway to take care of small load variations, and three 50 cusecs

syphon spills identical to those used at the control at the settling chamber. By these means the water level is maintained in the forebay within 1-½ in from full to zero discharge.

Space has been provided for the addition of a fourth syphon to be added when the power plant is expanded.

The penstock is fitted at the top with an automatic butterfly valve, which closes in the event of water flow exceeding 10% of the maximum required by two wheels at full output. An 18 in. air inlet stand pipe is located immediately below the butterfly valve. The penstock consists of a series of 26 ft. pipe elements, having diameters of 42 and 48 in., chosen so as to permit nested shipment from the factory. The joints are welded and four expansion joints have been provided.

The penstock terminates in a Y laid in the anchor block connected to the turbine connection flanges through electrically operated gate valves. The anchor block is tied in solidly with the power house foundations.

In the absence of rock in this area it was found necessary to carry the foundation excavations down to a strata of well-compacted sand and gravel, with some boulders, just on the limit of the water table.

The two power units consist of Gilbert Gilkes & Gordon 33 in. single-nozzle, 600 r.p.m., 3,300 h.p. "Turgo" type turbines, with jet deflectors controlled by oil-pressure governors. Each turbine drives a General Electric Company 2,500 kW, 3,300 V alternator and direct connected exciter. Each generator feeds its own transformer through 3·3 kV switchgear.

A voltage of 33 kV has been adopted for the transmission system. No circuit breakers are provided at this stage between transformers and line, but these are contemplated when future expansions will render necessary the duplication of the transmission circuit.

The transmission line presented a difficult problem. Observations of uncertain accuracy made in previous years indicated a probable isoceraunic level in excess of 100. The length involved covers some 21 miles, so that it was obvious that protection must be provided to reduce the shutdowns for lightning flash-overs. To achieve this, the 33 kV system was solidly grounded with earth resistance at both ends of the line less than one ohm. An H frame wood pole construction with three-element glass suspension chains provided a high impulse strength, and two overhead wires were run with earths at each pole, so that it is hoped that the number of lightning outages may not exceed two or three a year.

Of particular interest are the earths at the various poles; these consist of two or more cast iron pipes let into the ground to depths of 6 to 8 ft., to which have been brazed the galvanised steel down droppers. African experience has shown that copper or galvanised steel rods of the normal type tend to corrode rapidly in the lateritic soils, and build up resistances of high value; whilst cast iron pipes have proved to be reasonably lasting.

Lightning arresters are provided at all terminals. The transmission system is designed to transmit the output of the entire Mobuku power house with the extra units installed, maintaining a good regulation with a load power factor of 95 to 98%.

In the Kasese area a 2½ mile spur line supplies the roasting plant, and is tapped off from the main transmission by means of isolators that enable sectionalising to be carried out either towards the roaster or towards the mine. The spur line has four instead of three suspension elements, to obviate the possible serious effects of sulphur fumes and dust on line insulation. Line conductors are of Silmalec, an aluminium alloy with approximately the same electrical and mechanical characteristics of A.C.S.R., only easier to handle, being of uniform formation.

The receiving station for the roaster comprises two 750 kVA 33/3·3 kV transformers, star-delta connected and solidly earthed on the high tension side. The transformers are fed through fused disconnects without high-tension circuit breakers. All the distribution protection and switching is done on the low tension side by means of a very comprehensive 3·3 kV switchboard.

The substation for the mine and mill is in all ways similar to that at the roaster, except that the transformers are two 2,500 kVA units, duplicates of those at the generating station; voltage taps serve to differentiate between sending and receiving end voltages.

The entire project cost £627,000, or £97 per h.p., a very reasonable unit cost considering the distant location of the site and the lack of any construction facilities excepting those brought in for the development of the mine.

Here following is a percentage breakdown of the cost.

Intake	12·5%
Flume	38·0%
Forebay	3·0%
Penstock	9·0%
Power House	21·5%
Transformer Substation ...	1·6%
Transmission Line	13·0%
Communications	1·4%

When the power house expansion has been completed and the power output is increased to 9,000 h.p. the unit cost will be even less.

Further developments connected with this hydro-electric project have already been the subject of extensive investigations, and cover the diversion of the Asa into the existing forebay and the building of a second plant using the tail water of the present one. The new power house would be located on the banks of the Sebwe River and could generate some 8,000 h.p.

Engineering in Action. A publication under this title has been issued by the Allis-Chalmers Industries Group and gives a general review of the organisation's achievements during the year 1956. Reference is made to the 79,000 h.p. turbines being made for the St. Lawrence River Project and to the 75,000 h.p. six-jet Pelton machines being constructed for the Donnells Dam Project in California.

Bruce Peebles & Co. Ltd., Edinburgh, have obtained a £250,000 order for three 90,000 kVA transformers for the Magill sub-station of the Electricity Trust of South Australia. The transformers are for connection to the Adelaide end of the new 275 kV transmission lines from Port Augusta power station and will feed into the Trust's 66 kV and 11 kV metropolitan systems. Weighing about 200 tons each in their completed operating condition, the transformers will be the largest three-phase units so far to be installed in Australia. The specified impulse level is 1,050 kV and the transformers will be tested accordingly on the company's new 2,400 kV impulse generator.

Original Tendencies in Power-House Design in Hydro-Electric Installations - I

By SERGE LELIAVSKY, Ph.D., M.I.C.E. A.Am.Soc.C.E.

BY the end of the last century there was a wide variety of types of turbine arrangements, the choice among them being frequently dictated by the personal opinion of the designer. For instance, contemporary writers quote the radial type of Fourneyron, the Henschel-Jonval axial-flow turbine, and many other types designated by the names of their inventors.

At that time, the general layout of the power house depended on the type and arrangement of the turbines installed, and therefore varied within very wide limits, thus making it impossible to trace general principles controlling the design of the structural works in the hydro-electric developments of the period. As time went on, however, and the capacities of the newly built power stations increased, these variations were gradually given up, and the first three decades of the present century witnessed a process of standardisation in turbine types, which culminated in almost standard design rules, as expanded in many modern books on the subject. In so far as structural works were concerned, this process inevitably led to the standardisation of the layout and dimensions of power houses, so that by about 1935 the arrangement commonly employed became, if not a rigid standard, at least a very frequently used type design.

Since then, however, new tendencies have become apparent, aiming at the following general objectives:—

(a) To reduce the curvature of streamlines, in order to minimise the losses of energy, and thus, staighten out the flow of water in the power house.

(b) Rearrange the power house in such a manner as to make it capable of serving the double purpose of (1) housing the turbines with generators, etc., and (2) evacuating the surplus discharge during flood. This last requirement is of particular importance when, owing to the specific configuration of the terrain at the site of the works, the width of the channel does not suffice to locate separate units serving each of these two objectives separately.

The diagrams in Figs. 1 to 7 can serve as simple examples, illustrating the principle of these general tendencies.[1] In Fig. 1 we have a classical type-arrangement for low and medium heads, which conforms with the standard practice as described in the modern textbook, and may therefore serve as a basis for comparison in examining other and less orthodox arrangements represented in other diagrams of the same group.

Attention will next be called to Figs. 2 (a), (b) and (c). The flow in (a) and (b) is obviously better and the general layout simpler than in the traditional type of layout shown in Fig. 1. This change in the design might have therefore resulted in a saving, had it not been for the cost of the crane required for erection and dismantling purposes, which in this case must command a rather wide area, and is consequently rather expensive.

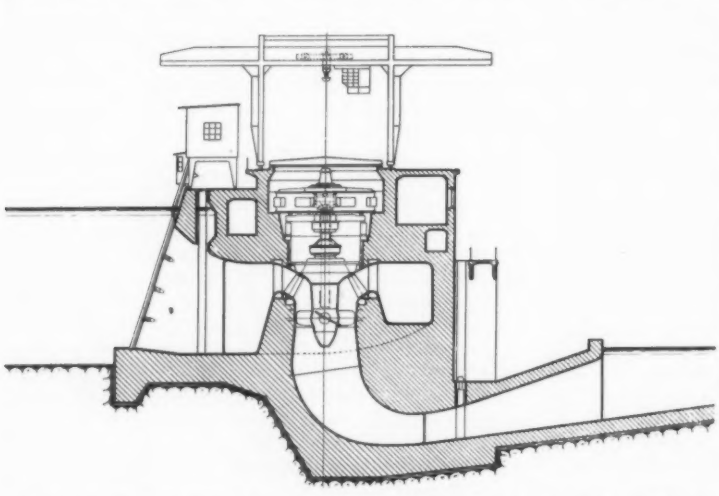


Fig. 1. Conventional low-head layout using vertical-shaft turbine

In this respect Fig. 2(c) is a material improvement. The bevel-gear transmission shown in this drawing is submerged in an oil bath, which is enclosed in the hub of the guide-vane wheel. It is evident that the satisfactory running of such a turbine installation depends entirely on the efficacy of the seals in the joints of the oil bath, stuffing boxes, etc., and if this problem is capable of a satisfactory solution, there is no reason why a more ambitious objective could not be attempted; namely, to place the entire generator in that same hub.

This problem was first solved in 1936, at the Rostin power station in Pomerania, by Escher Wyss Ltd. in collaboration with Arno Fischer. This turbine, known as the "Arno Fischer Turbine," is shown in Fig. 3. It is, of course, a rather small unit, the discharge being 6·3 cu. m. per sec. and the head 3·75 m. only. The generator receives cooling air through the fixed vanes of the guide wheels, which, for this purpose, are made hollow. Condensed vapour and the inevitable minimum of leakage are evacuated in the same manner.

The corresponding layout of the power station, as incorporated in the weir structure, is shown in Fig. 4. The cost of the station is low and the two objects (a) and (b) (see above) are both satisfied.

The next step in the evolution of this type was the power station built by the same firms on the Iller River in Bavaria. In this case (Fig. 5) the magnet ring of the generator was designed as a fixed part of the outer rim of the runner of the turbine.

The main problem of the mechanical design of such a combined unit depended on the high speed of the ring in question (20 m. per sec.) and on the practical importance of a watertight seal between this rapidly moving ring and the fixed parts of the turbine casing. Design data for the performance of the seal under such exceptional conditions, e.g. friction resistance, wear, etc., were not available when turbines of this new type were first designed. It was consequently necessary to obtain these data from special *ad hoc* tests.

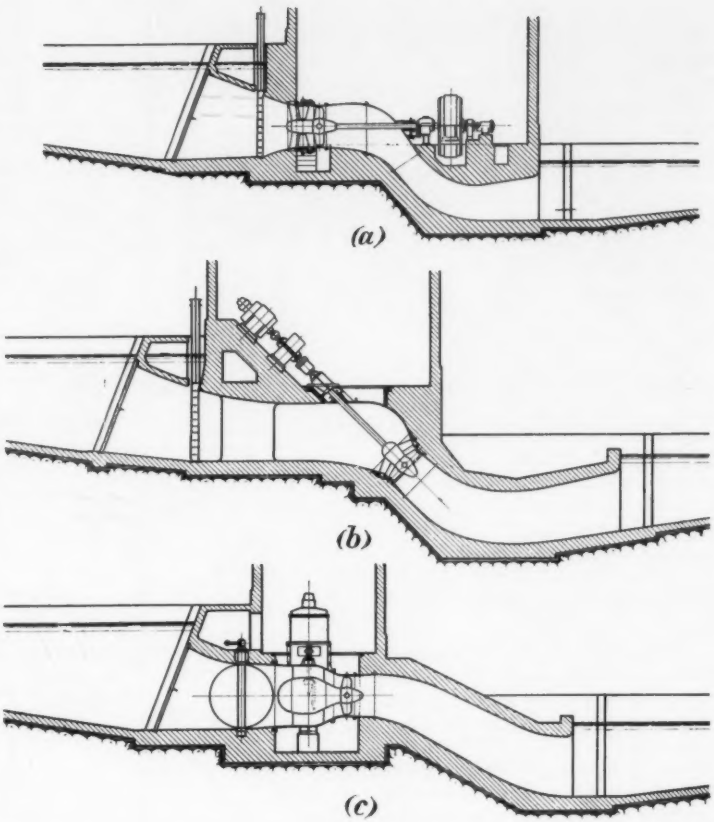


Fig. 2. Low-head layouts using horizontal or inclined-axis turbines

Fig. 6 shows the section of the power station incorporated in the weirs of the Lech River, while in Fig. 7 we have another development of the same principle, as used on the Saalach River.

In the one case the axis of the turbine is horizontal, whereas in the other it is inclined, the slope varying from 1 : 2 to 1 : 2·5. Anticipated erection difficulties for this sloping type have not materialised in practice, while from the hydraulic point of view it appears to possess definite advantages.

[1] These figures are abstracted from "The Tubular Turbine," by H. Cardinal von Widdern, in *Escher Wyss News*, Vol. 25/26, 1952/53, pp. 22-33—a periodical published by the firm of turbine makers, Escher Wyss Ltd., Zurich, Switzerland.

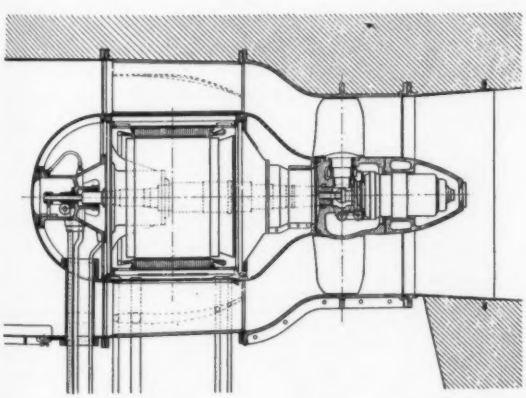


Fig. 3. Escher Wyss axial-flow turbine with generator arranged in the guide-wheel boss, at Rostin power station, Pomerania

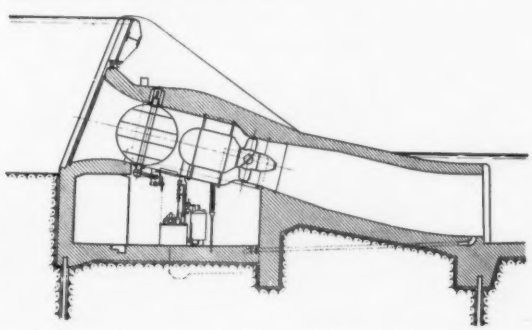


Fig. 4. Submerged power plant at Rostin with tubular turbine

The head and discharge for this type of turbine are respectively of the order of 8 m. and 20 cu. m. per sec. The hub, blades and outer rim of the runner are a single steel casting, with the magnet ring of the generator shrunk on it. The outer diameter of the blades attains 2·15 m. The installation as a whole is, therefore, a rather small undertaking which may nevertheless be cited as a type convenient for minor or medium rivers or for canals on a distributary irrigation network.

An attempt to apply a similar principle to a much larger installation is shown in Fig. 8. The drawing shows a section proposed for the Foster Creek dam, one of the several alternative designs studied in 1943 by the U.S.A. Corps of Engineers as a part of a vast programme of hydro-electric developments on the Columbia River, downstream of the Grand Coulee dam.[2]

It is rather curious to observe how the change in the size of the projected work alters the scope of the main problems of its structural design; in fact, while in the earlier solutions, reproduced in these pages for smaller works, the designer's attention was focused on questions of general convenience, such as facility of erection and the like, the main problems to be resolved in structures of a different order of magnitude, as exemplified by the Foster Creek Dam project, are the stresses and structural stability of the main dam. It goes without saying that the usual dam-design standards based on the assumption of linear stress

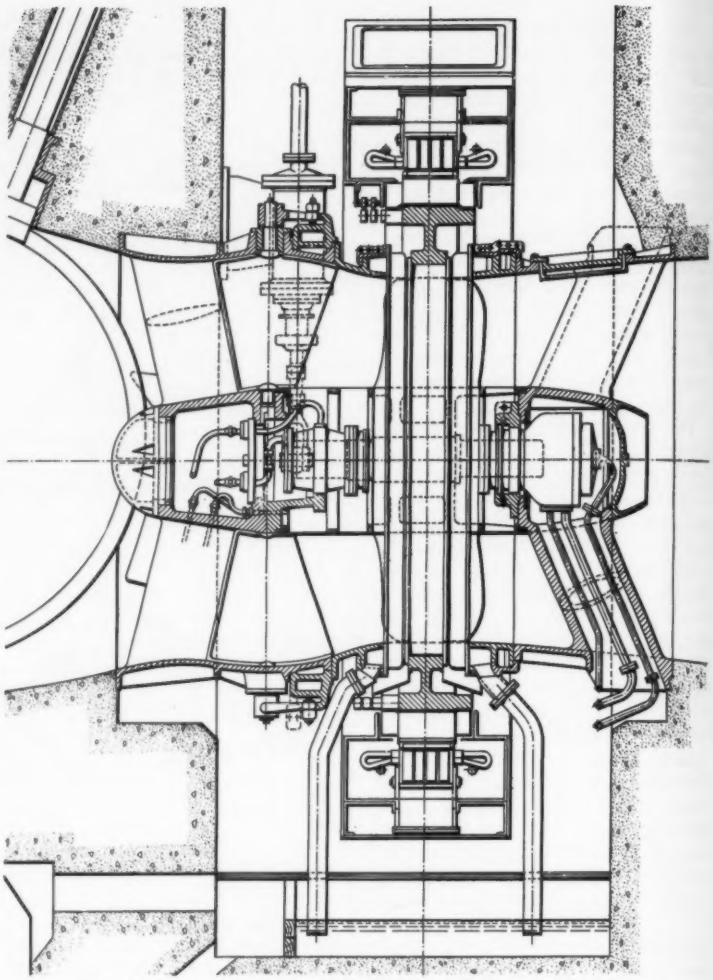


Fig. 5. New design of Escher Wyss tubular turbine

[2] The drawing is reproduced from the paper on "Photoelastic Analysis of Stress in a Dam containing a Large Gallery" by R. L. Sanks, published in the *Proceed. Am. Soc. Civ. Eng.*, as Separate No. 693, in May 1955. The writer is indebted to the same publication for Figs. 8 to 12 incl. as well as for other information on the same subject.

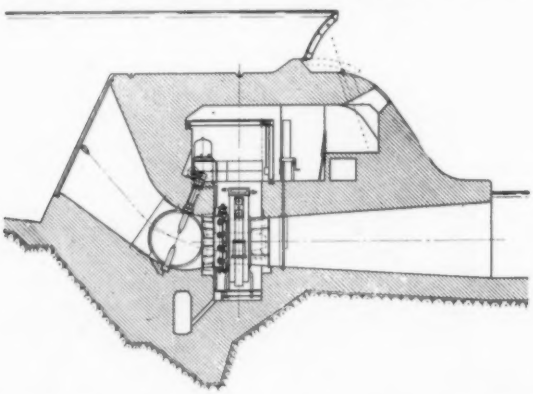


Fig. 6. Submersible-weir power plant with tubular turbine arranged horizontally to facilitate erection

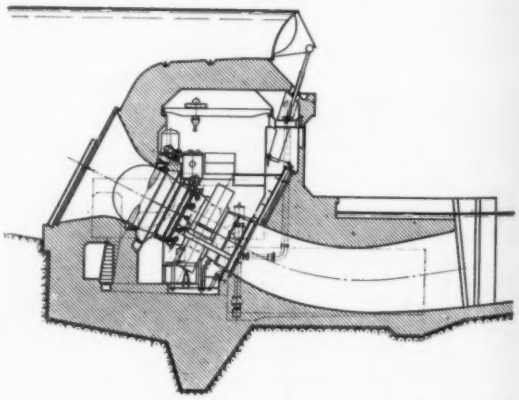


Fig. 7. Submersible power station at Saalach with inclined tubular turbine, affording a short inlet and better guiding of the incoming water

distribution, e.g. the middle-third rule, etc., do not apply in the case under consideration, because the very large gallery, provided in this dam to house the turbines and generators, alters basically the pattern and type of the concrete skeleton. As this problem, apart from its intrinsic significance in connection with the particular project investigated, bears also on all other structures of the same general layout, a special investigation was carried out by means of the photo-elastic method, and deserves quotation in these pages.

Photoelasticity, as such, is obviously beyond the scope of this general review,[3] but without going into the details of the relevant theory, the results of the investigation under consideration are summarised below in terms easily understood by the average designer.

The effect of the structural weight on the one hand and that of the hydrostatic pressure applied to the face of the dam on the other were dealt with separately by means of two different models and experimental arrangements. In the first case, the model was rotated, gravitational effects being thus replaced by centrifugal forces. The average centrifugal force in the centre of the model was 280 gravities (900 r.p.m.).

[3] It is dealt with in numerous excellent treatises, e.g., Coker and Filon's "A Treatise on Photo-Elasticity," Cambridge, 1931.

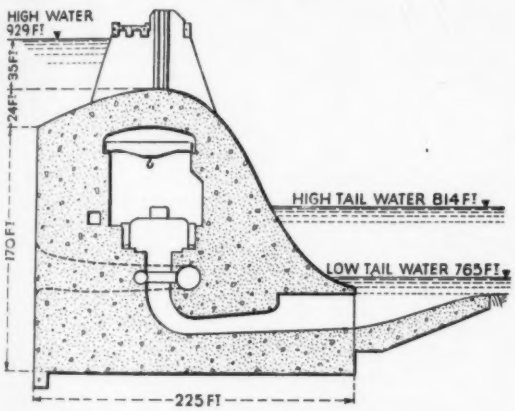


Fig. 8. Project for Foster Creek dam

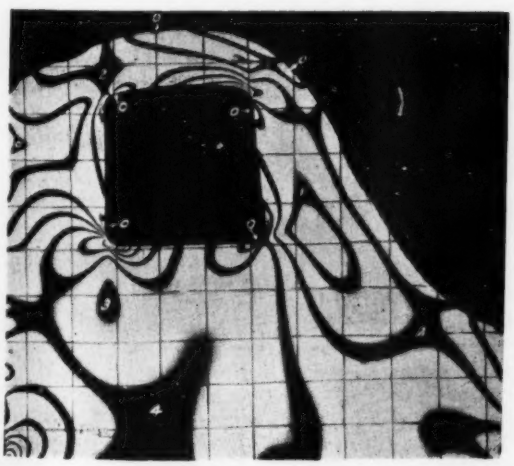


Fig. 9. Fringe photograph for structural weight

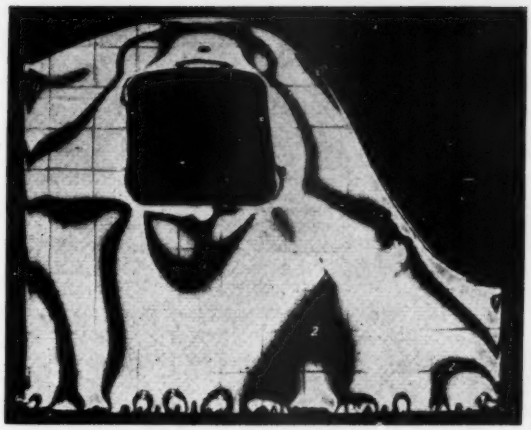
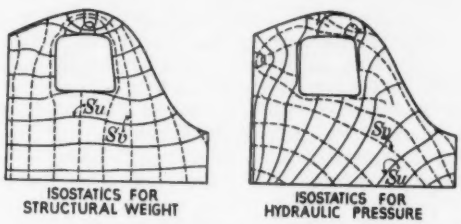


Fig. 10. Fringe photograph for hydraulic pressure



Figs. 11 and 12. Isostatics for Foster Creek dam models

The stresses obtained in this manner were "frozen" into the model—a particular type of photoelastic technique, developed in the last 15 years or so, in which the photoelastic effect remains even after the load is removed. To this end the model was placed in an oven, which rotated with it. The temperature was then increased to 118°C. in a 2½ hour period, held constant for 3½ hours, and again decreased to 28°C. in a 16 hour period, the rotating speed being kept constant from the start to the end of the test. Since the characteristics of a centrifugal field differ from those of a gravitational field, the results obtained in this experiment had to be adjusted accordingly, which, however, was a relatively simple mathematical operation of more or less the usual type, requiring no particularly difficult solution.

On the other hand, in studying the stresses due to the "live load"—i.e., in this case the water pressure—the hydraulic effect was replaced by a device consisting of a "stack of beams," which distributed the mechanical load applied over eight pads conforming as closely as possible with the principle of hydrostatic pressure distribution.

One of the most obvious characteristics of the photoelastic method is the "fringes," or black bands, appearing in a model strained in a field of monochromatic polarised light, which are the loci of all points of some particular value of the difference of principal stresses. Fringe photographs obtained for the two models used in these experiments are shown in Figs. 9 and 10 respectively.

While they constitute the outstanding item of the information supplied by such tests, fringe diagrams

alone are not enough to calculate the stresses in all the points of the profile. They must be supplemented for this purpose by other information, e.g. isoclinics, which are dark bands appearing in a strained model in a field of plane polarised light. These bands are the loci of all points at which the direction of the principal stress corresponds to the axis of polarisation.

In addition to such purely photoelastic data, measurements of lateral strains yield the sums of principal stresses, thus contributing towards a greater precision of the final result. In this manner all the stresses can be obtained, yielding as they do comprehensive information for design purposes. For instance, Figs. 11 and 12 show the isostatics, i.e. principal-stress trajectories, computed in this manner for the two models in question.

Combining the stresses created by the two causes contemplated in these experiments, tension in the concrete was found to exist only at the top and bottom right-hand corners of the gallery. It is left to the designer's choice either to use steel to take these tensions or to modify the profile in such a manner as to prevent the tension from occurring altogether.

(To be continued)

Book Reviews

La Moderne Turbine Idrauliche (Kaplan, Francis, Pelton) ed i Regolatori di Velocità. In two volumes: *Testo* (text) 918 pp., 1,066 ff. and *Atlante* (14 tables and 30 plates). By Dr. Ing. Giacomo Büchi. Published by Ulrico Hoepli Editore, Milan, Italy. Paper covers. Price complete, 15,000 lira (about £8 12s. 6d. or $20).

This is a book to warm the turbine designer's heart. After seeing so many textbooks on standard hydraulics in which turbines are dismissed by the conventional treatment of types of turbine, blading velocity triangles, the standard formulae, and if we are fortunate, a note on specific speed, we find it a pleasure to turn to a book that is obviously the outcome of a lifetime of practical experience in turbine design. We sense this immediately on reading the opening chapters, for the standard subject matter on the flow of water in conduits and within blades is quickly dismissed in favour of the examination of flow conditions, vortex formation, etc., by means of flow nets, and by blade calculations based on the conception of the space between blades as a closed channel. Throughout the book there is this emphasis on the practical hydraulic design problem as it confronts the turbine engineer. The book is divided into six main sections: Elements of Hydromechanics as applied to Turbines; General Theory of the Reaction Turbine; Construction of Reaction Turbines and Typical Installations; The Pelton Turbine; Governing of Turbines; and Accessories (bearings and lubrication, and shut-off valves).

The volume of tables and plates is an acquisition in itself, for it contains tables giving leading particulars of most of the larger turbine installations in Italy, Europe, the Americas including Canada, and to a lesser extent in other parts of the world. The numerous plates are devoted mainly to layout drawings of important installations.

Dr. Ing. Giacomo Büchi, who died in 1947, was chief engineer of the Riva company of Milan for many years, and in 1923 became technical director of the associated firm of Calzoni of Bologna. He spent the last five years of his life in the preparation of this book, but publication was not immediately possible. The text has now been co-ordinated and brought up to date by his son, Dr. Ing. Riccardo Büchi, and is published for the first time. A foreword has been contributed by Dr. Ing. Guido Ucelli of the Riva company. The book, of course, is in Italian.

Boysen Dam and Power Plant. Report issued by The United States Department of the Interior, Bureau of Reclamation, Denver, Colorado, 1957, 226 pp., 89 ff.

This report gives a complete technical account of the design and construction of the Boysen dam and power plant and is one of a series of similar records which are being made of the Bureau's undertakings. The works are situated on the Bighorn river, central Wyoming, and consist of an earth and rockfill structure having an open-channel spillway on the left abutment capable of dealing with 100,000 cusecs. The power plant is located on the right bank and has a capacity of 10 MW in two units.

It forms part of a multipurpose scheme and has three definite objectives: to provide much-needed power in the more or less immediate area, to irrigate some 100,000 acres of new land, and to regulate floods. It will afford recreational facilities.

The scheme is not exceptional in any way and the interest of the report lies in the detailed account that is given of the design and construction of the earth dam, the grouting programme, the spillway, the power plant structure and its hydraulic and electrical equipment, and in the elaborate cost figures that are reproduced and which cover about 40 pages of text.

International Electrotechnical Vocabulary, 2nd Edition. Published by the Central Office, I.E.C., 1, rue de Varembe, Geneva, Switzerland. This publication, No. 50 (15), refers to Group 15, i.e., switchboards and apparatus for connection and regulation, of the Second Edition of the International Electrotechnical Vocabulary which will contain 22 groups when complete. It contains some 260 terms and definitions in French and English, together with the equivalent terms in Dutch, German, Italian, Polish, Spanish and Swedish.

The terms and definitions are divided into the following sections:—General terms; types of construction and physical protection; constructional elements; operation; characteristic quantities; circuit-making and circuit-breaking devices; mechanical relays; fuses; plugs, sockets and lamp-holders; apparatus for starting and control; apparatus for protection against surges or over-currents; switchboards, cubicles and boxes; accessories for electric wiring; power station accessories.

W. Ralston Limited, the industrial photographers of Glasgow, are this year celebrating their 100th anniversary. We have frequently made use of their views in the various articles we have published describing the schemes promoted by the North of Scotland Hydro-Electric Board.

New Range of Compaction Rollers

MANY construction projects which involve the use of earth in the foundation or subsequent stages, require the material to be consolidated, often to a considerable degree. To meet the growing demand for suitable equipment for this purpose, Vickers-Armstrongs (Tractors) Limited are now manufacturing different types of compaction rollers, designed primarily for use with their own well-known range of heavy crawler tractors. Two types of rollers are produced, the Sheepsfoot and the Rubber-tyred Compactor, and between them pretty well all variations in soil-compaction requirements can be met.

The Model 2-157 "Compacta" Sheepsfoot Roller is produced in one size, a normal arrangement comprising two individual units joined by articulated members. Other arrangements of 1, 3 or 4 units are available.

Each unit consists of a drum, which can be filled with water-ballast, running on a fixed axle attached to a rectangular frame on which sand or earth ballast-boxes are mounted. Tapered feet welded to the periphery of the drum are arranged in 36 rows each with four feet, and staggered to give eight lines of compaction. Each foot is nine inches long and has a hard alloy steel pad welded to the end. Two types of feet are available, either oval in section $6\frac{1}{2}$ in. by $4\frac{5}{8}$ in. tapering to 3 in. diameter, or 5 in. square tapering to $2\frac{1}{2}$ in. square. The roller with the square-shaped feet is known as the Model 2-157 Mark II.

The addition of water and/or sand ballast enables ground pressures of 330 to 920, or 368 to 1,026 p.s.i. to be obtained, this versatility allowing the correct pressure to be selected for each particular application. The articulated joints permit the units to move independently when coupled together as a pair, allowing the complete roller to conform to the surface irregularities which occur during the initial stages of compaction.

The uniformity resulting from this arrangement is one of the most advantageous features of the roller; without it, soft patches would occur which might pass unnoticed, only coming to light when heavy loads had been applied for some time or when subsidence resulted.

Multi-unit Arrangement

Multi-unit arrangement can be adopted when required; the basic 2 unit model can be used with the individual rollers arranged as a pair, or in tandem by the use of an additional towbar. A further one or two units can be added when necessary to meet any particular operating requirement.

The Vickers Onions rubber-tyred compaction roller is produced in two sizes, each comprising a group of independent units enclosed within a rectangular yoke. Each unit comprises a wheel and tyre, supporting a container designed to carry sufficient sand ballast to load the tyre to its safe maximum. An adjustable towbar enables the roller to be kept in trim when towed behind a suitable tractor, while the yoke may be altered to take from three to six units.

The compaction units are hinged to the front of the yoke and can move independently. At the rear, each unit is attached to a buffer spring to damp excessive oscillation. These springs preserve the stability of the assembly without transferring any appreciable load from one unit to another.

By the towing arrangement at the front of the yoke with the stabilising springs at the rear, a high degree of uniform compaction is obtained. Each unit is free to move vertically, with the tyre imposing a steady and even load over ground irregularities. The consolidation provided by this roller eliminates soft patches, particularly when dealing with materials which include small rocks.

The yoke on each size of roller can be altered, by different combinations of adjustable spacing bars, to accommodate from three to six independent units, the standard model being equipped with four. The total weight of the four-unit Model "75" when fully ballasted with wet sand is 50,500 lb., giving a load of 760 lb. per inch of tyre rolling-width. For the four-unit Model "150," the equivalent figures are 105,724 lb. and 1,185 lb.

Flexibility

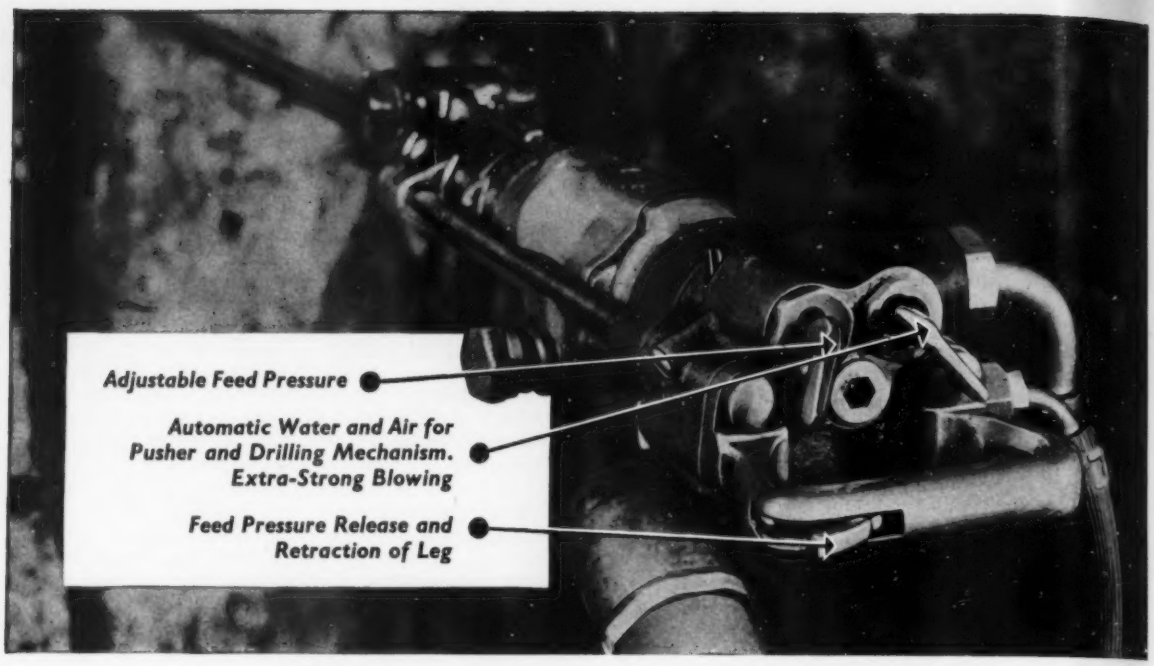
The requirements of projects involving earth consolidation at some stage of the operations differ considerably. Due to the large number of variable factors involved, no two jobs will be identical, and this aspect of compaction was borne in mind when designing both types of roller; in consequence, each model has a high degree of flexibility as far as the ultimate ground pressures are concerned. At the same time, the standard models can be adapted to meet different conditions by the addition of extra units.

TABLE I

Type of soil	Moisture percentage	Dry density consolidated in lbs. per cu. ft.
Heavy clay	25	105
Silty clay	20	110
Sandy clay	15	110
Sand	10	120
Gravel with sand and clay	9	130

Compaction results in the reduction of air-voids and the removal of excess water from the soil, stability depending upon conditions of cohesion and friction between the soil particles. No amount of compaction will, however, give "permanent" stability to sand or gravel; rock or clay fill must therefore be introduced to produce a mixture that will stabilise. As compaction proceeds, the resistance increases towards the point of refusal at a rate depending upon the composition and water-content of the soil.

Moisture-content is a vital factor and can be either too high or too low. It particularly affects the sheepsfoot roller, which is rendered ineffective when too high a moisture-content causes it merely to puddle the ground. For this reason, the employment of the sheepsfoot roller is limited in countries where the nature of the climate maintains a high moisture content.


Adjustable Feed Pressure
Automatic Water and Air for Pusher and Drilling Mechanism. Extra-Strong Blowing
Feed Pressure Release and Retraction of Leg

THE ATLAS COPCO LION— A REVOLUTIONARY NEW ROCK DRILL

The drill for heavy duty work

All controls under one hand

The Atlas Copco Lion is the first drill to have all the valves which operate the drill under the control of one hand. *Full and easy control without having to move the hand from the backhead!* All the controls have been designed so that they are well protected. While using them the operator's hand is never near the wall or roof of the drift. The Lion is the first pusher leg drill with controls placed for drifting.

Retractable leg saves time

When the leg has to be moved the feed pressure is easily released by squeezing the hand grip. By further pressure on the grip the leg retracts automatically. When the leg is in the new position suitable for continuous drilling, retraction stops and the feed pressure comes back by loosening the grip of the hand. *All this can be done while the drill is still running.*

This new idea of a retractable leg enables quicker repositioning of the leg and reduces the number of steel changes, thereby increasing footage per manshift. When drilling high holes it is now far easier to alter the position of the leg more frequently in order to maintain an optimal feed angle and feed pressure.

Packed with power for deep holes

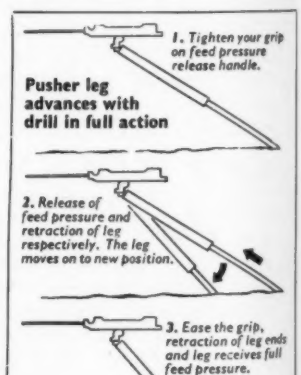
The Lion has a drilling rate at least 30% higher than other rock drills of the same weight. Furthermore, it can maintain its high speed *even when drilling deep holes.* This means quickly drilled deep hole rounds and a faster advance. The Lion also reduces *to a minimum* the gauge wear of the bits in abrasive rock. And owing to the ease with which the feed pressure is released and brought back into action, the Lion is a *handier* drill to work with in fissured rock.

Sandvik Coromant—the steel for the Lion

All Atlas Copco drills—and this goes for the Lion too—have been developed from the earliest stages with Sandvik Coromant tungsten-carbide-tipped integral steels and detachable bits. This combination pioneered tungsten carbide drilling in the early forties. No drill or steel developed separately could ever give such equivalently high performances as this drilling combination. Today it is the most widely used in the world, responsible for drilling more than 1,000 million feet annually.

World-wide sales and service

Atlas Copco puts compressed air to work for the world. It is the largest group of companies specialising solely in the development and manufacture of compressed air equipment. It embraces Atlas Copco companies or agents manufacturing or selling and servicing Atlas Copco equipment in ninety countries throughout the world. For further details of the equipment featured here, contact your local Atlas Copco company or agent, or write to Atlas Copco AB, Stockholm 1, Sweden.


1. Tighten your grip on feed pressure release handle.
Pusher leg advances with drill in full action
2. Release of feed pressure and retraction of leg respectively. The leg moves on to new position.
3. Ease the grip, retraction of leg ends and leg receives full feed pressure.

Atlas Copco

DSS74R

Manufacturers of Stationary and Portable Compressors, Rock-Drilling Equipment, Loaders, Pneumatic Tools and Paint-Spraying Equipment

TABLE II.—SPECIFICATION DATA FOR RUBBER-TYRED COMPACTION ROLLERS

Individual Compaction Units	TYPE 75		TYPE 150	
Tyre size	14·00 × 20 in. × 16 ply		18·00 × 24 in. × 24 ply	
pressure	80 lb./sq. in.	5·62 kg./cm.²	90 lb./sq. in.	6·33 kg./cm.²
Maximum permissible load on tyre*	16,020 lb.	7,260 kg.	32,550 lb.	14,764 kg.
Rolling width of tyre	1 ft. 4·6/10 in.	0·42 m.	1 ft. 10·3/10 in.	0·565 m.
Maximum permissible rolling speed*	10 m.p.h.	16·09 km./hr.	10 m.p.h.	16·09 km./hr.
Ballast container: length	13 ft. 1 in.	3·99 m.	16 ft. 8½ in.	5·09 m.
height	4 ft. 7 in.	1·40 m.	5 ft. 9 in.	1·75 m.
width	1 ft. 8¾ in.	0·53 m.	2 ft. 3½ in.	0·70 m.
capacity	75 cu. ft.	2·12 m.³	158·8 cu. ft.	4·49 m.³
Complete Compaction Rollers	**MODEL 4-75**		**MODEL 4-150**	
Total weight: without ballast	14,500 lb.	6,577 kg.	29,559 lb.	13,381 kg.
fully ballasted†	50,500 lb.	22,907 kg.	105,724 lb.	47,956 kg.
Load per inch of tyre-width fully ballasted†	760·5 lb.	345 kg.	1185·2 lb.	537·6 kg.
Overall dimensions: width	7 ft. 9½ in.	2·37 m.	10 ft. 4 in.	2·15 m.
length including towbar	23 ft. 3 in.	7·08 m.	26 ft. 10½ in.	8·19 m.
height to top of containers	5 ft. 9 in.	1·75 m.	7 ft. 1 in.	2·16 m.
height to top of spring rods	6 ft. 10 in.	2·08 m.	8 ft. 6 in.	2·59 m.
Turning radius	20 ft. 3 in.	6·17 m.	22 ft.	6·71 m.
Ground clearance fully loaded (approx.)	11 in.	0·28 m.	1 ft. 2 in.	0·36 m.
Drawbar pull required: on level ground	8,000 lb.	3,629 kg. (estimated)	15,000 lb.	6,804 kg. (estimated)
5% gradient	12,000 lb.	5,443 kg.	20,000 lb.	9,072 kg.
heavy going	16,000 lb.	7,257 kg.	22,000 lb.	9,979 kg.

* Maximum load per tyre at stated pressure to be reduced when rolling speed exceeds 10 m.p.h. (16·09 km./hr.)
† Based on wet sand at 120 lb./cu. ft. (4,237 kg./m.³): for dry sand reduce figures by 16·7%.

3, 5 or 6 unit rollers are available in both sizes. Different adjustable spacing bars are available for fitting on the yoke, to accommodate a greater or lesser number of individual units.

A pneumatic roller on the other hand works satisfactorily with a higher moisture-content, the existence of which enables the soil particles to "float" into place. For each type of soil there is a point of optimum water-content at which consolidation takes place most speedily and completely. It is not infrequently an advantage to add water to the soil, by means of hoses or sprinklers, until sufficient is present to plasticise the mass into its most workable condition.

Table I shows approximately the optimum moisture-content and the maximum dry-density of the more common types of material.

Choice of Compaction Unit

From the foregoing it will be apparent that the choice of compaction roller for individual projects will be governed by the material to be consolidated, the operating conditions at site, and the specifications of the contract. An outline of the performance characteristics of each type may however be of assistance in

TABLE III.—SPECIFICATION DATA FOR SHEEPSFOOT COMPACTION ROLLERS MODEL 2-157 COMPACTA FOR USE WITH TRACTORS OF 15,000 LB. DRAWBAR PULL

	MARK I		MARK II	
DRUM DIMENSIONS				
Length	5 ft. 10 in.	1·78 m.	5 ft. 10 in.	1·78 m.
Diameter (less feet)	5 ft. 0 in.	1·52 m.	5 ft. 0 in.	1·52 m.
Diameter (with feet)	6 ft. 6 in.	1·98 m.	6 ft. 6 in.	1·98 m.
Number of feet	144		144	
Number of feet per row	4		4	
Centre to centre distance between feet in row	1 ft. 6 in.	45·7 cm.	1 ft. 6 in.	45·7 cm.
FOOT DIMENSIONS				
Base at drum	6½ × 4⅛ in.	16·5 × 11·74 cm.	5 × 5 in.	12·7 × 12·7 cm.
Length	9 in.	22·8 cm.	9 in.	22·8 cm.
Area of foot face	7 sq. in.	45·1 sq. cm.	6¼ sq. in.	40·1 sq. cm.
OVERALL DIMENSIONS				
Length (including towbar)	19 ft. 3 in.	5·87 m.	19 ft. 3 in.	5·87 m.
Width	13 ft. 8½ in.	4·17 m.	13 ft. 8½ in.	4·17 m.
Height (on hard ground)	6 ft. 6 in.	1·98 m.	6 ft. 6 in.	1·98 m.
WEIGHTS (for complete two-drum unit)				
Empty	18,407 lb.	8,349 kg.	18,407 lb.	8,349 kg.
Drums water ballasted	30,900 lb.	14,015 kg.	30,900 lb.	14,015 kg.
Ballast boxes with wet sand at 120 lb./cu. ft.	38,847 lb.	17,620 kg.	38,847 lb.	17,620 kg.
Drums and ballast boxes loaded	51,347 lb.	23,290 kg.	51,347 lb.	23,290 kg.
GROUND PRESSURES				
Empty	330 lb./sq. in.	23·2 kg./sq. cm.	368 lb./sq. in.	25·9 kg./sq. cm.
Drums ballasted	550 lb./sq. in.	38·7 kg./sq. cm.	618 lb./sq. in.	43·5 kg./sq. cm.
Boxes ballasted	695 lb./sq. in.	48·8 kg./sq. cm.	776 lb./sq. in.	54·6 kg./sq. cm.
Drums and boxes ballasted	920 lb./sq. in.	64·7 kg./sq. cm.	1,026 lb./sq. in.	72·1 kg./sq. cm.

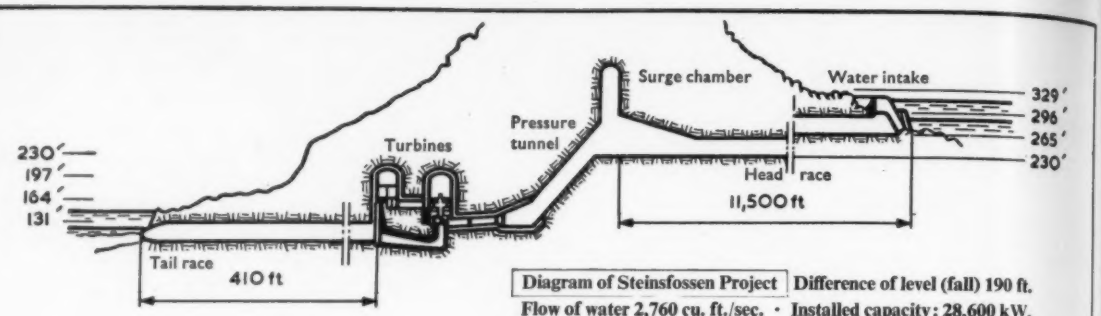


Diagram of Steinsfossen Project | **Difference of level (fall) 190 ft.**
Flow of water 2,760 cu. ft./sec. · Installed capacity: 28,600 kW.

Atlas Copco equipment at Steinsfossen

Shifts of only 8 men removed 2,300 cu. yds. per week in a 540 sq. ft. tunnel

AT THE HYDRO-ELECTRIC PLANT at Steinsfossen in Norway the contractors, A/S Høyer-Ellefsen, have chosen Atlas Copco compressed air equipment and Sandvik Coromant drill steels for all rock excavation. The head race tunnel at Steinsfossen, 11,500 ft. long and with a 540 sq. ft. section, has shown how swift, economical and simple tunnelling can be. Each week the tunnel was drifted 115 ft.—a particularly remarkable achievement for the number of men employed. This steady advance went on uninterrupted for week after week. Three shifts were worked daily *with only eight men doing all the tunnelling work*—drilling, blasting, mucking, and transportation. They were supported by two mechanics and one dump-clearer to a shift.

World-famous drilling unit

All drilling at Steinsfossen is done with Atlas Copco light rock drills and Sandvik Coromant integral steels. Specially developed over many years to work together, these drills and steels are now the most widely used drilling unit in the world. The ease and efficiency of this equipment is best illustrated by the fact that at Steinsfossen *eight men kept nine drills working simultaneously* at the face.

Speedy Atlas Copco LM 200 Loaders

At Steinfossen 240 cu. yds. of rock of big fragmentation were mucked-out per hour (effective loading time) by two LM 200 Atlas Copco Loaders. These loaders are particularly efficient because they are compact and narrow. Being railbound the minimum of space is required for manoeuvring and two can be used side by side at the face even in a relatively small tunnel. The loaders are also used for moving the drilling platform as the work proceeds. They are simple to operate and do not call for skilled workers.

Dual purpose drilling platform

Vital to the ease and speed of the whole operation was the Vinstra drilling platform which enables miners to drill from three levels. It has two movable beams on which the cars can be hung and swung across from one rail to the other. On the average car-switching took one minute.

Facts about the Steinsfossen operation

Nature of rock: Granite — Head race tunnel: 11,500 ft. long, 540 sq. ft. section

Approx. advance per year: 4,900 ft. — Average advance per week: 115 ft. (128 ft. max.)

Personnel per 24 hours:

Foreman	1
Miners	24
Mechanics	6
Assistants	2 (rail laying, ventilation, air and water pipes, etc.)
Dump-clearers	3
Total:	36

Drilling equipment: 10 Atlas Copco rock drills type RH656 with pusher legs (one spare). Sandvik Coromant drill steels.

Loaders: 3 LM 200 (one spare). Loading time (including car-switching) 3.25 minutes per car of 4.4 cu. yds. capacity, i.e. 47 cu. yds. solid rock per hour and loader.

World-wide Sales and Service

The Atlas Copco Group embraces Atlas Copco companies or agents manufacturing or selling and servicing Atlas Copco equipment in ninety countries throughout the world. For further details of the equipment featured here, contact your local Atlas Copco Company or Agent or write to Atlas Copco AB, Stockholm 1, Sweden.

Atlas Copco *Manufacturers of Stationary and Portable Compressors, Rock-Drilling Equipment, Loaders, Pneumatic Tools and Paint-Spraying Equipment*

determining which is most suited to particular applications.

The design of the sheepsfoot roller is such that the projecting feet consolidate upwards from below, and in its passage over the fill, the whole weight of the roller, whether ballasted or otherwise, is concentrated in turn on each row of four feet. This results in high concentrations of pressure on small surface areas which after a number of passes over the fill cause the feet to "roll themselves out of the ground." To obtain a homogeneous mass, therefore, the fill should be compacted in layers no thicker than the length of the roller feet, thus enabling them to consolidate to the full depth of each layer.

In this manner, an unlimited number of layers of fill can be placed, and consolidated to uniform density throughout. This form of compaction, as frequently used on dams, embankments and road sub-bases, possesses the added advantage of the towing tractor's ability to use its dozer blade for spreading heaps of dumped earth, without interruption of rolling.

By comparison, the rubber-tyred roller is capable of dealing with thicker layers of material, as the compactive pressure is exerted to a greater depth. This roller also achieves a high degree of compaction where stones and rocks are intermixed with the soil, and is particularly successful on materials which have a tendency to flow under pressure.

The high tyre-loadings and inherent stability of the roller also enable it to be operated at relatively high speeds, a factor of great importance where large quantities of fill are being placed by other equipment in restricted working areas.

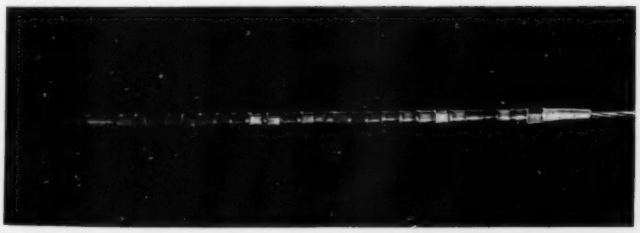
Materials having a high moisture-content can be efficiently consolidated as a result of the "kneading" action of the rubber tyres, the excess moisture being forced to the surface to be dispersed by evaporation. Owing to soil friction and resistance, lower particles of material tend to become slightly less compressed than the upper surface, which develops a closely-knit "skin" capable of shedding rainwater to a remarkably high degree.

New "Uniojint" for ACSR

A new tubular compression joint suitable for communication and distribution line work has been developed by the Aluminum Company of Canada Limited and Aluminium Laboratories Limited. Called the "Unijoint," since it comprises a single sleeve only, it can be used up to and including No. 4/0 AWG 6/1 ACSR size. The new principle also is applicable to dead ends, called "Uniterms."

Compared with former joining methods, the Unijoint is claimed to be not only lower in first cost but also, as the installation is greatly simplified, to cheapen line-erection costs. To make a joint, the two ends of the conductor are inserted into the sleeve, which is indented at the centre to eliminate measuring or guessing at the position of the joint. The sleeve is compressed with a mechanical hand tool ,and the joint is complete.

The Unijoint is prefilled with a new type of jointing compound which greatly increases the friction between the sleeve, the conductor strands and the steel core wire, so that joints made with a single sleeve develop the rated ultimate strength of the conductor.



Top left: A "Unijoint" in a run of cable; bottom left: A dead end with jumper being clamped; right: Clamping tool in use

LOYNE DAM

The smaller of the two dams constructed by The Mitchell Construction Company as part of the North of Scotland Hydro-Electric Board's Glen Moriston Scheme. In its construction precast facing blocks developed by the Company were used, providing yet another modern constructional technique.

Consulting Engineers: Sir Wm. Halcrow & Ptnrs., M.I.C.E.

THE MITCHELL CONSTRUCTION COMPANY

WHARF WORKS PETERBOROUGH

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The prefilled sleeves are easier to handle, and the lineman does not have to carry a container of compound. Uniform application of the compound is assured, and the joint is automatically sealed after compression. Small plastic caps over the ends of the sleeve keep the compound in place before the sleeve is installed.

Joints and dead ends made with the Unijoint and Uniterm were tested extensively by Aluminium Laboratories Limited to determine the effect of temperature cycling and relaxation on the mechanical strength of the joints. Following 300 hours of temperature cycling, tensile tests showed no noticeable loss of strength. The electrical resistance of a conductor joined with a Unijoint, before and after current cycling, is well below that of an equal length of unjointed conductor.

Abstracts from the World Technical Press

Hydro-Electric Construction in Switzerland

A comprehensive survey is given of the present trends in the design of water-power plant in Switzerland, proceeding from a review of the development of energy consumption in the country since 1910. Some typical hydro-electrical installations are described and their characteristic features discussed. A map shows all the water-power plants and groups of plants which were in operation in 1956; their aggregate output now exceeds 20 million kWh a year. (Prof. G. Schnitter, Federal Technical University, Zurich, *Technische Rundschau*, Vol. 49, No. 17, April 26, 1957, p. 1, 5 pp., 20 ff.)

Progress at Peruča Dam, Dalmatia

This rockfill structure, 420 m. long and 60 m. high, will create on the Cetinia River a power storage of 500 million cu. m., covering an area of 18 sq. km. at storage level. The nature of the foundation rock made extensive grouting imperative and the watertight curtain extends to no less than 1,700 m. in length and 200 to 250 m. in depth. It consists of a triple row of grout holes 150,000 m. in aggregate length. The construction site is well organised and abundantly equipped with modern plant ensuring a high degree of efficiency. The diversion of the Cetinia River into the diversion tunnel was completed in the middle of June 1956. The rockfill, when being placed, is sluiced with water under pressure. Work on the watertight clay core was started at the beginning of October 1956. (Ing. Vjekoslav Krajič, *Energia*, formerly Bulletin of the Croatian Electricity Union and Zagreb Electricity Institute, Vol. V, January-February 1957, No. 1-2, p. 18, 5 pp., 2 ff.)

U.S. and Swiss Dam Designs

This is a comparative study by an American civil engineer on dam design criteria in U.S. and Swiss practice. Actually, on the Swiss side, there is but one structure instanced as a basis of comparison, viz. Grande Dixence. Except when dealing with stage construction, the two American structures taken into consideration are the Hoover and Shasta dams. In the paragraph devoted to stresses, the Swiss designer is considerably bolder than the American. For instance, while the gravity section of Hoover dam, carrying less than the full hydrostatic loading, has a base width equal to 91% of the height, the corresponding figure is 77% at Grande Dixence which is designed to carry full hydrostatic loading. At the Shasta dam, another straight gravity section with a base equal to its height (600 ft.), if the slopes were projected to the 922 ft. height planned for Grande Dixence, the base would be 996 ft., i.e. 40% wider. Also the maximum computed stress at the heel of the Shasta dam was 586 p.s.i. and at the toe 744 p.s.i.; the corresponding figures at Grande Dixence are given as 1,067 and 1,849 p.s.i. The maximum calculated stress at the downstream toe of Grande Dixence is given by Hunerwadel, its chief construction engineer, as 1,849 p.s.i., which is double the present permissible U.S.B.R. standard. When considering resistance to sliding, one examination customarily made in design is a comparison of the ratio of horizontal forces to vertical forces; this ratio is calculated to be less than the angle of friction of the material. In a triangular dam section with water at the apex of the triangle, as at Grande Dixence, this ratio is independent of the height of the dam. The ratio is increased when uplift is included, and is still higher when earthquake forces are included. The assumed magnitude of this force, as used in the design, was not stated; however, by using Westergaard's method and an acceleration of 0·01 gravity, horizontal forces exceed the vertical forces when uplift is included. To prevent the ratio from being in excess of the angle of friction when earthquake effect is taken into account, the design of any high straight gravity dam is revised from a triangle with a vertical upstream face to a non-vertical upstream face or a broken slope section, as was done at Shasta dam. The additional weight of the upstream sloping section and the vertical component of the water loading on the sloping section improves the ratio. Also the overturning moment due to water loading is decreased by using an upstream slope. The above comment on stress distribution applies to a monolithic and homogeneous structure. It is, however, proposed to build Grande Dixence first to a height of 584 ft. and later raise it to 922 ft. Such an instance of stage construction implies the use of the reservoir before the completion of the dam, so that some blocks will be stressed by partial water loading prior to the placing and grouting of the adjacent unloaded blocks. This procedure alters the assumption of the section being a monolith. Methods proposed to be used for making adjustments in stress due to stage construction of Grande Dixence were not stated. On the American side, Marshall Ford dam in Texas was the only instance of a straight gravity dam exceeding 200 ft. in height; it was raised from 204 to 278 ft. by adding an upstream slab or wall grouted to bond with the original upstream face after the wall was completed. This is no precedent for raising a 584-ft. high straight gravity dam to 922 ft.; design details of how the Swiss engineers propose to bond various sections into a homogeneous monolithic structure would be a welcome contribution to high gravity dam design.

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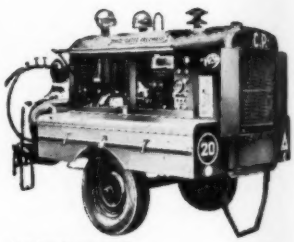

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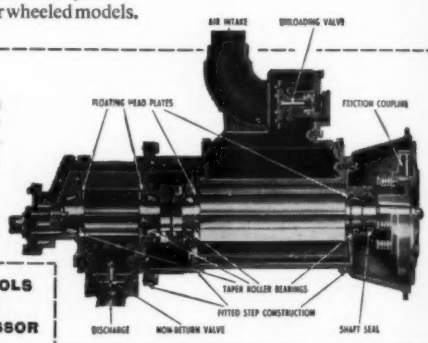

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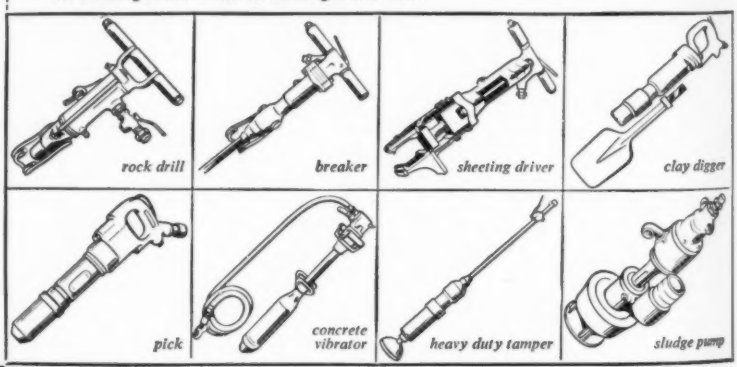

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In the reply made by Mr. Hunerwadel after reading this article, the Swiss expert draws attention to the difference between the sites of the Shasta and Grande Dixence dams; while the Shasta dam is erected at a point where the valley is quite open, the Grande Dixence site is in a very narrow V-shaped valley. It may also be said that the conditions imposed by the Swiss government have a specific value for Grande Dixence and cannot be applied as a generalisation to all Swiss designs. As to stage construction as planned for Grande Dixence, Mr. Hunerwadel refers to paper C-34 presented at the last Conference of the International Commission on Large Dams in Paris 1955, and published by Commission Internationale des Grands Barrages, 91, rue St.-Lazare, Paris 9. (W. H. Holmes, Civil Engineer, Sacramento, Calif. *Engineering News-Record*, Vol. 158, No. 13, March 28, 1957, p. 32, 4 pp., 2 ff.)

Power Development in West Bengal

In this brief article the author surveys the power development possibilities in West Bengal, with special reference to hydro-electricity, and points out the necessity of an integrated development programme of the rich water resources available for power and irrigation. The State's coal reserves are also considerable, and the two together should provide immense possibilities for meeting the rapidly increasing power demand in a region whose industrial development programme is growing every year. (Dr. M. Datta, *Indian Journal of Power and River Valley Development*, Vol. VI, No. 11, November 1956, 6 pp., 4 ff.)

Developing the International Stretch of the Douro

The 780-km. long Douro or, as it is called in Spain, Duero, is one of the major streams of the Iberian Peninsula, draining an area of 98,000 sq. km., that is to say about one sixth of the total area of the peninsula. In its 145-km. middle stretch, the river forms the border between Portugal and Spain and, according to an agreement concluded between the two countries, the downstream half is to be developed by Spain and the upstream half by Portugal, the dividing point between the two sections being the confluence with the Tormes torrent. This article deals with the Portuguese section which, as planned by Hidouro (Hidroelétrica do Douro), is to be equipped with a coordinated chain of three plants, Miranda, Picote and Bemposta, of which the middle step, Picote, now in an advanced stage of construction, is described in detail. The main features of this plant are: (1) a concrete four-bay overflow arch dam of the symmetric double-curvature pattern, fitted at the crest with 20 m. by 9 m. tainter gates designed for a maximum flood discharge of 10,000 cu. m. per second. The downstream end of the ski-jump type spillway is supported by five buttresses structurally independent of the dam proper. On the right bank, a 5·5 m. diameter reinforced concrete tunnel, with an inlet 35 m. below the storage level of the reservoir, will serve as an auxiliary spillway. (2) The intake works, also on the right bank, consist of three 5·5 m. by 7 m. portals protected by trashracks, each connecting with a penstock 5·5 m. in diameter. (3) The underground machine hall, 85 by 16·6 m., which entailed about 40,000 cu. m. of excavation work, and houses three 60 MW vertical Francis-driven generating groups. The draft tubes, each fitted with a control gate, discharge into a common tailrace tunnel 170 sq. m. in cross-section, which returns the water to the river at a point just upstream of the dissipation zone of the ski-jump spillway. The surface installations are erected on the slope covering the machine hall, about parallel to the river, and comprise: the control building, outdoor switchyard and substation, and an erection hall which connects with the machine hall by a shaft and short tunnel. Work at Picote began in the first months of 1955, and the plant is scheduled to go into service early next year. A full description of the construction plant is also given. (*L'Energia Elettrica*, Vol. XXXIV, No. 2, Feb. 1957, p. 175, 7 pp., 10 ff.)

Penstock Concreting in Inclined Shafts

This article describes the application of the "Prepakt" American method as used at Kemano, B.C. and Kaprun, Austria. The method is first explained and discussed, and the concreting of the Kemano and Kaprun penstocks is then described in detail in separate chapters. Kemano characteristics: penstock 3·3 m. dia., made of plates varying between 14 and 48·4 mm. in thickness, and placed in an inclined shaft 1,162 m. long by 4·2 m. in dia. The penstock, made of welded lengths varying between 8·4 and 16·8 m., was so laid in the shaft as to leave a space of 60 to 120 cm. between steel plate and rock, which was then packed with aggregates up to 75 mm. maximum size. Aggregate packing was made by "rock blowing" in the horizontal sections and, in the inclined sections (48°), with a view to suppressing dust, by means of a spiral-welded steel pipe 25 cm. in diameter and fed at the top from a hopper. Subsequently, the pipe was concreted in, together with the penstock. Once the aggregates were in place, grout was injected from inside the penstock through series of six holes provided around the wall, beginning with the bottom holes in the horizontal sections. In the inclined sections, grouting was effected by means of a train of three travelling platforms driven by a winch; the grout was conveyed, according to the distance, by two or three relays of pumps, first to the mixer on the middle platform, then to the bottom platform, whence two pumps pressed it back, each through a separate pipe, to the grouting platform at the top. There each delivery pipe bifurcated so that four grouting holes could be fed at a time. Grouting started through a special hole leading to the bottom layer of placed aggregates, and as soon as grout appeared in other holes of the same series, the grout injectors operated into those holes. At Kaprun, the characteristics of the inclined shaft were as follows: length 441·19 m., angle of slope 51° 15 ft., and the diameter 3·3 to 3·5 m. The penstock, 2·7 to 2·9 m. in diameter, had a wall thickness from 28 to 18 mm. and was laid in sections 10 in. in length. The theoretical volume of concrete involved was 2,800 cu. m. The aggregates used for the prepacked concrete varied between 15 and 80 mm. Concreting proceeded as follows: from a storage silo at the top of the shaft, and immediately after a penstock section was put into place, a 2 cu. m. truck was lowered by means of a 5 ton hoist as far as the upper edge of the penstock section, and emptied its load of aggregates through a sliding door. There was no vibrating of the penstock sections and grouting proper began

only after a further penstock section had been placed and welded, and the aggregates poured. The grouting mortar was fed from the mixer at the top of the shaft to a hopper from which a 5 cm. dia. pipe transferred it by gravity to a tank fitted on the distribution truck. The latter was connected by a 5 cm. dia. pipe and by a telephone link with the grouting platform. The vertical distance between distribution truck and platform was determined to correspond with a pressure of the grouting mortar not to exceed the maximum compressive strength of the penstock wall, i.e. 5 atm. Grouting was effected by means of three rubber hoses leading from the 5 cm. distribution pipe 3·75 cm. dia. holes in the penstock wall, starting with the bottom holes. As soon as grout appeared at the top holes, the distributing system was shifted to the next grouting point. This method produced excellent results. It should be noted that, for the five uppermost sections, that is to say when the penstock line had reached a point where the distance between distributing truck and grouting platform was no longer sufficient to ensure the required pressure of 5 atm., grouting was effected by means of a pump. (Waldemar de Wetlinsky, Ing. I.E.T., *Construction*, Vol. XI, No. 10, October 1956, p. 366, 6 pp., 9 ff.)

Head Losses in T-Branch Pipes

This paper deals, in two instalments, with the tests carried out by the author at the Hydraulics Laboratory of the Lausanne Technical University with a view to determining the pressures occurring in each of the three branches of a T conduit having various modes of distribution of the flow among the three branches and with different patterns of T connection as to diameter and angle. The importance of these tests is emphasised by the fact that they can be applied to determining the influence of the kinetic energy of water in the power conduit on the oscillations in surge chambers. The testing installation consisted of a straight main pipe divided by a branch pipe into an upstream and downstream section. Eleven various patterns of connection were investigated in two series; one in which the diameter of the branch pipe varied between 150 and 60 mm., the diameter of the main pipe remaining uniformly 150 mm., and the other in which the diameter of all sections of the T assembly remained constant at 150 mm., but the angle of the branch connection with the main pipe varied between 45° and 135°. Two patterns of flow were investigated: (1) splitting the discharge of the upstream section of the main pipe between the branch pipe and the downstream section; (2) leading into the downstream section of the main pipe a discharge equal to the sum of the discharges of the upstream section and of the branch pipe. The measurements were effected by means of diaphragms of the type prescribed by V.D.I. standards. The tests enabled the head losses under various conditions to be determined. A general interpretation of the results by the analytic method would scarcely be possible; however, in the case of the discharge supplied entirely by the branch pipe, the analysis attempted by Professor Fabre has been used as a basis for empirical formulae of general applicability by the author who, in particular, worked out roughly the law governing head losses in the case of split discharges. On the other hand, the results obtained have also been applied to the computation of surge chambers, and proved that mass oscillation is slightly weaker when effective head losses are taken into account. Contrarily, the governing stability of a hydro-electric plant is secured to a lesser extent than could be accepted when taking into consideration the effect of the kinetic energy of water exactly at the point where the surge chamber is inserted in the system. The stabilising effect may frequently be reduced by 30% and, in some instances, more. (A. Gardel, *Bulletin Technique de la Suisse Romande*, Vol. 83; No. 9, April 27, 1957, p. 123, and No. 10, May 11, 1957, p. 144; 15 pp., 23 ff.)

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